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Preparation of Technical Papers*

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SYNOPSIS

The author deplores the inadequate training of many engineers in English composition—their poor sense of literary form, their halting and ineffectual attempts to express themselves with precision and clarity. He quotes many published examples of the faults he finds; suggests improved passages, and further, on the constructive side, recommends a course of study.

A military engineering office in the West recently called upon the local federal civil service headquarters to find an engineer with editorial experience. The salary offered was high compared with the proverbially limited government pay for technical service. To the few who applied for the position a civilian interviewer explained:

"We have had exceptional difficulty in obtaining engineers who can write reports in a form suitable for publication or submission to Washington. Our experience has shown that the average engineer knows nothing about writing. Much that is sent in to this office from the field is almost pitiful in its poverty of form. Those of us who do the rewriting find the work a severe strain; one member of the force has taken a leave of absence because of high blood pressure brought on by the strenuous work."

As illustration of the deficiency the interviewer showed samples of field reports which resembled rambling high school themes.

Papers presented in engineering magazines and bulletins offer ample evidence of failure of many leading American technical men to express ideas with satisfactory clearness and completeness. It is upon the writings of those engineers whose experience and study have made them authorities that the profession relies largely for knowledge of standard methods and up-to-the-minute progress. These accepted authorities

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have earned recognition by devoting their lives to the practical mechanics of their work; they have, as a rule, spent all their time in acquiring knowledge of mathematics, materials, labor problems, and continuous changes in practice to the exclusion of the finer arts such as writing. As a result many of America's best engineers are handicapped in this essential work of communicating ideas.

It would be far from true to say that there have been no technical men with ability for interesting and lucid presentation of observations. Every structural designer of mature age is acquainted with J. A. L. Waddell's "Bridge Engineering" of three decades ago, a set of two large volumes that reads like a novel in its masterly presentation of the subject. Instances of well-planned and clearly presented contributions to engineering literature in recent years are the numerous exact and comprehensive studies on structural steel by the late Leon S. Moisseiff, noted structural engineer. Many present-day authors deserve commendation. But it is a misfortune that so many of America's scientific men, who as a class enjoy unquestioned rank among the world's best, should appear mediocre in one branch of their activity.

British engineering journals show that English engineers look to America, sometimes enviously, to study standard practice, working conditions, and salaries. We in turn might well look back across the Atlantic to study our neighbors' superior use of words. A difference in literary tendencies between these two peoples with common speech is to be expected. The British have cultivated their native English for centuries with the result that accumulated refinements in expression have passed down from father to son. Likewise other Europeans have a unified heritage of speech. We Americans, however, have no common language background. Our population is derived from all races of Europe and each race has brought its own vocabulary. In the shift to English the heterogeneous people have not been able to assimilate the new language as completely as they might absorb a native tongue. We see at times disappointing evidence in the result such as the excessive slang and intentional misuse of words that hint at disrespect for the adopted language.

Into our textbooks, reports, journals, and popular books on science goes a great deal of confused, half-thought-out wording. Ambitious salary workers who spend evenings in study waste much time glaring at pages and wondering what the writer intended to say. It should be axiomatic that if a writer expects his words to be read he should accept the responsibility of making them unquestionably clear at a glance and not leave sentences that the reader will have to puzzle over. The reader should not be expected to ponder over the meaning of words while he is

thinking of the ideas themselves. As a rule engineering concepts, as such, are easily comprehended by an educated and experienced engineer. Grasping of knowledge is not difficult to a mind that understands the facts upon which the knowledge is based. Study of the printed page usually is necessary only to ferret out meanings of obscure passages. That thoughts can be conveyed concisely and efficiently to the reader is proved by newspapers in which journalists present the most complex national and international problems so that even the partially educated masses read the accounts with interest and rapidity of comprehension and feel that they have knowledge of the world at their fingertips.

It is not the intention of this paper to discourage any engineers, industrial leaders, or other technical men from writing for publication. By all means a structural designer who has worked out an improved method of stress analysis or a manufacturer advocating a new use for concrete should give his findings and opinions to the world even though he has had no writing experience at all. In his verbal presentation the amateur cannot be expected to show the exactness of John Henry Newman or the ability for detail of Robert Louis Stevenson, nor will he use the interest-catching devices of the journalist, but he can at least give his paper the thought necessary to arrange ideas in an orderly fashion and present words which most exactly fit the intended meaning.

Let us consider some details in the preparation of a technical paper. The writer can further clearness at the start by explaining in the first paragraph why the subject discussed is worthy of presentation. Such a statement when used as justification of a news account is known to journalists as the "feature" or "lead." Two examples of possible openings of this type are:

> This paper is the first description presented to the public on the use of glued scrap-lumber laminated arches.
>
> When Navy engineers tore down sections of a quay wall in salt water they found reinforcing bars completely rusted through. In the new concrete sea wall, described here, special coverings protect the steel.

From such an introduction the reader learns what the article contains and judges whether it will be of interest to him.

If the paper describes an engineering project, a summary giving a complete picture will prevent the necessity of the reader struggling later to piece together the component parts. This summary corresponds to the plot plan of a set of architectural or structural drawings. Such a description might be:

> The project consists of 250 duplex dwellings; office building; Imhoff tank with sewage pumps; water, gas, sewer, and power lines;

3 acres of grading and sodding for lawn; 8000 sq. ft. of concrete sidewalk; and 2.5 miles of street grading and pavement.

As the writer later discusses each item in turn the reader expects it and understands without undue thought its proper relation to the whole.

Wording throughout the article requires most conscientious care. Too often a writer hastily accepts inaccurate wording that comes to his mind, assuming that because it has for him an association with his idea, it will also have the same association for the reader. In this respect writing is a sort of self-delusion. The author-to-be creates the subject matter in his mind and after a period of reflection embellishes it with exact mental details. The ideas are now all perfectly clear. He will put them into words. He composes a sentence that recalls to him the first mental picture. He writes the sentence. A second sentence recalls another mental picture; he puts that down, and so on. After completing the article he reads it over. The words remind him of all the ideas he intended to present, the ideas that were already clear and fixed in his mind. Good, he says, writing that recalls such vivid pictures must be perfect.

The author offers the paper to a friend to read. The friend looks at it. His face registers perplexity. What does this sentence mean, and this one? The author feels annoyed by the stupidity of the friend whom until then he considered intelligent. He explains with emphasis that it is all perfectly clear; the words mean thus and thus. Oh, yes, now the friend understands, after it is explained to him.

General and abstract ideas are the most difficult to describe. Concrete objects and actions appealing to the senses, on the other hand, can be pictured more effectually. Let us take this sentence as an example:

A large white horse galloping down the center of the 20-foot-wide asphalt pavement stumbled and fell in a heap.

Here familiar objects and concrete actions which in real life would be directly perceivable by the senses present a clear and forceful picture. But it is difficult to impart to others knowledge that cannot by its nature be reduced to sense perception and simple basic actions associated with our lives. In selecting words to use as the means of expressing such general ideas, the writer must take pains first to decide exactly what he wants to say and then to say exactly that. In the struggle to find the elusive right word the thesaurus is a great help, and no writer, even the most accomplished, can work satisfactorily without this handbook.

A study of engineering publications and textbooks provides ample evidence of faulty expressions. A few taken from the printed pages of current

literature will illustrate the tendencies toward inexactness. The first is from a well-known publication dealing with concrete design:

> For each of the four types (of haunch shapes in beams) the shape of a member is characterized by two *ratios*, *a* representing the length and *b* the depth of the haunch. The value of $b = \frac{\text{min. I}}{\text{max. I}}$ is assumed to equal the ratio of $\left(\frac{\text{min. d}}{\text{max. d}}\right)^3$, the beam width being considered constant and reinforcement disregarded.

The idea intended by these two sentences is simple, but it is given only partially. Every engineer who has read the publication has had to stop and study the inferences that follow this quotation to learn what ratio *a* is. It would have been as easy to present the first sentence as follows and allow the reader to grasp the thought immediately:

> Two ratios determine the characteristics of each of the four types: *a*, the length of the haunch divided by the total length of the beam; and *b*, the moment of inertia of the maximum haunch divided by the moment of inertia of the beam.

In the second sentence the words "the ratio of" are unnecessary.

This ending of a paragraph is from a popular book on science:

> As no ghost can talk to a biologist, so no biologist can talk to a spiritualist. Biology, spirituality; materialism, mysticism; natural, supernatural; faith in the present, faith in and hope for the future; science, religion.

Here is the case of a writer erroneously assuming that words which recall an idea to him will necessarily present the idea to others. Apparently the late noted anthropologist had a definite message on the contrast between religion and science, perhaps a statement that the two modes of thought are irreconcilable; but exactly what he wished to convey we cannot ascertain from the words.

In an interesting detailed report on the repaving of a highway by laying a brick course over the existing concrete the project engineer included a contradictory statement:

> As the new crown did not correspond to the old (which was very irregular), the elevation used depended on the crown of the old slab at any point.

From the controlling data given, one would judge that the writer wished to say:

> As the new crown did not correspond to the old, which was very irregular, the elevation used depended largely on the peaks of the old crown.

The report continues:

> The elevations were plotted to a scale of 1 mm., or .01 ft., and where the existing crown was more than the new, the center elevation was plotted. Otherwise the highest edge was plotted.

The description of scale is not clear. "More" probably refers to elevation.

> The elevations were plotted to a scale of 1 mm. to .01 ft. Where the existing crown was higher than the new, the center elevation was shown; otherwise the highest edge was plotted.

This sentence is from an account of the manufacture of plywood:

> Wartime demand for logs has decreased by about 2 feet from the 5- to 6-foot minimum diameter formerly accepted.

The writer starts with one sentence and ends with another. Though the meaning is apparent, the words actually indicate that the demand for logs has decreased. Deletion of "from" will correct the defect.

> Wartime demand for logs has decreased by about 2 feet the 5- to 6-foot minimum diameter formerly accepted.

The meaning of the following sentence is doubtful:

> Consider a symmetrical beam with straight haunches, dimensions and loads being as shown in Fig. 2.

At first glance the reader assumes that "straight haunches, dimensions and loads" forms a series; then he decides that probably "dimensions and loads" starts a separate, second phrase. No doubt the writer meant:

> Consider a symmetrical beam with straight haunches, the dimensions and loads being as shown in Fig. 2.

Long unbalanced sentences hinder free flow of thought. The quotation below is from a discussion of strength of spaced wood columns in an authoritative publication on timber design.

> When blocks are placed and calculations for permissible loads are made as recommended, safe unit loads for spaced columns result (so far as strength or resistance to lateral buckling in a direction perpendicular to the mutually contacting surfaces of the blocks and individual members is concerned) and it is unnecessary to check the unstayed portions (from center of bolt in end blocks to center of spacer block) of the individual members.

The first parentheses set aside an essential idea that should be prominent. This example demonstrates, by the "are placed" and "are made" of the first clause, another practice general among technical men, the free use of the passive voice in verbs in predominating positions. Professional authors, however, know that forceful writing requires, when feas-

ible, the active voice in leading verbs. Rearranging this obscure sentence will make it clear.

> Calculations of permissible loads for spaced columns with blocks placed as recommended do not need correcting for possible buckling in a direction perpendicular to the contacting faces of the spacers, and it is unnecessary to check the unstayed portions, from the center of bolt in end blocks to center of spacer block, of the individual members.

An engineer who had observed flood damage made valuable conclusions on the causes and possible preventive measures, but he stated a finding erroneously:

> I think that much of the flood damage is due to the fact that our citizens really do not know how high their land is above maximum flood level.

Knowledge of how high land is above maximum flood level will not help in preventing damage. Such land is safe. It is the area below high water and depth of possible inundation that is a measure of potential destruction.

> I think that much of the flood damage is due to the fact that many of our citizens really do not know to what extent their land is below maximum flood level.

An article on the strength of concrete construction joints had defective syntax.

> The fundamental principle of the sand cure method of joint cleanup was to place the type of concrete and in such a manner that the surface which formed one face of the joint would be equal in quality to the concrete mass and then protect and maintain that surface in the same condition as it was when placed until the next concrete lift was placed.

The conjunction "and" joins the noun phrase "the type of concrete" and the adverbial phrase "in such a manner that . . . " leaving awkward sentence structure and confused thought. Rearranging will improve the sentence.

> The basis of the sand cure method of joint cleanup is the placing of concrete of suitable quality in such a manner that the surface forming one face of the joint will be equal in quality to the concrete mass, and then, until the next lift is poured, protecting and maintaining that surface in the same condition as it was when placed.

This is from an engineering magazine:

> Inlet and outlet structures in the canal system of the Imperial Irrigation District began to be made of precast concrete when war

necessity brought about restrictions in the use of lumber for such purposes.

It is true that the intended meaning is obvious. It is illogical, however, to say that "structures began to be made" because the beginning was made by the District, not the inanimate objects.

The Imperial Irrigation District changed the construction of inlet and outlet structures in its canal system to precast concrete when war necessity brought about restrictions in the use of lumber for such purposes.

Words and phrases of transition are often necessary to explain relations between successive ideas.

Following retaining wall construction, repairs were made to the road pavement. The smaller breaks were repaired as the wall was poured.

An added short explanation will eliminate the apparent contradiction.

Following retaining wall construction, the major repairs were made to the road pavement. The smaller breaks, however, were repaired as the wall was poured.

Webster's Dictionary does not give a meaning for the word "orient" that would conform to this use:

To orient on the specific work that was carried out, Table 1 has been prepared.

A substitution will eliminate the defect.

As a guide to the work that was carried out, Table 1 has been prepared.

In a description of the higher cost of Gunite work in New York a writer has offered this statement:

Work done in New York City requires that the nozzleman be a cement finisher and the gun be operated by a concrete laborer.

In skeleton form the sentence is: "Work . . . requires that . . . " Actually it is not the work that requires specially classified men; it is the labor union that makes the requirement. The work itself would demand no greater skill in New York than in any other city.

On work done in New York City there is a requirement that the nozzleman be a cement finisher and the gun be operated by a concrete laborer.

The following is from a discussion of analysis of bending moments in statically indeterminate structures:

There are perhaps two additional features that enter into structural design and analysis. These are: a. The necessity of taking

into account the often wide departure from theory dependent on a different stress distribution from that on which the theory is based.

The last sentence can be revised to provide more logical sequence of thought.

These are: a. The necessity of taking into account the fact that theoretical stress distribution upon which computed stresses are based may often be greatly erroneous.

This selection dealing with road alignment does not give enough information to indicate the intended meaning:

An attempt was made to keep the grade breaking in a given direction for a minimum of 100 feet, but during the latter part of the project this was reduced.

The word "unit" has become popular within recent years as a substitute name for a great variety of objects. For the sake of conciseness, however, a writer should not allow this word to replace repeatedly the exact term. Especially, two "units" with different meanings within one sentence will not make for greatest clarity.

Approximately 210,000 temporary family dwelling units with wall area aggregating about 180,000,000 sq. ft. and requiring more than 200,000,000 units have been or are being built of concrete masonry.

The first "unit" is unnecessary; "blocks" or "sections" would be more exact for the second.

Approximately 210,000 temporary family dwellings with wall area aggregating about 180,000,000 sq. ft. and requiring more than 200,000,000 blocks have been or are being built of concrete masonry.

It is true that the foregoing defects are extreme. Our technical literature, nevertheless, shows too much of such obscurity. Vagueness may not be, as in the examples, confined to one or two sentences: it may run through paragraphs or the article as a whole.

As to the question of raising the standards of American technical literature, it should be apparent that some steps need to be taken, such as stressing English in the curriculum of engineering colleges. The belief that English should play only a minor part, if any, in engineering courses is an unfortunate corollary to the present-day supposition that machinery and industry are the highest forms of intellectual achievement. It should be obvious, in this connection, that although some forms of engineering exist without higher mathematics, no technical work of any kind would be possible without language. Such instruction, however, should not wait for college; high school years are the most practical period for grounding the student thoroughly in English. In any case,

our language standards will not improve until there develops a general realization that English deserves more respect, more time, and more work than it receives now.

RECOMMENDED BOOKS FOR STUDY AND REFERENCE

Writing the Technical Report, J. R. Nelson, McGraw-Hill.

The Preparation of Reports, R. P. Baker and A. C. Howell, Ronald Press.

The Engineer's Manual of English, W. O. Sypherd and Sharon Brown; Scott, Foresman & Co.

Report Writing, C. G. Gaum, H. F. Graves, and L. S. S. Hoffman; Prentice-Hall.

Writer's Guide and Index to English, P. G. Perrin; Scott, Foresman & Co.

English and Engineering, Frank Aydelotte, McGraw-Hill.

The Art of Writing, Sir Arthur T. Quiller-Couch, Cambridge University Press.

Expressive English, J. C. Fernald, Grosset & Dunlap.

What a Word! A. P. Herbert; Doubleday, Doran & Co.

College Handbook of Composition; E. C. Woolley, F. W. Scott, and E. T. Berdahl; D. C. Heath and Co.

A Dictionary of Modern English Usage, H. W. Fowler, G. P. Putnam's Sons.

Roget's Thesaurus, Grosset & Dunlap.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1945 for publication in the JOURNAL for June 1945.

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Discussion of a paper by W. D. Bigler:

Preparation of Technical Papers*

By WILLIAM C. SPIKER†

The "Preparation of Technical Papers" by Mr. W. D. Bigler is, in the writer's opinion, a very important contribution to the advancement of engineering. The recommendations made are clear, positive and constructive. Until very lately the subject has been slighted.

For the purpose of helping to promote interest in the subject the writer makes the following suggestions:

The author of the paper, by adhering closely to the general construction of the sentences criticized has, while making the sentences technically correct, failed sufficiently to accentuate the economic waste resulting from involved, irrelavent, trivial and false statements.

Let us look at the problem from an engineering point of view. The value of a sentence is directly proportional to the value of the fact stated or the idea formulated; but, its value is reduced in direct proportion to the difficulty involved in understanding it.

The writer has borrowed two of the authors examples of poor writing to illustrate the point he wishes to emphasize:

First Example—"For each of the four types (of haunch shapes in beams) the shape of a member is characterized by two ratios, a representing the length and b the depth of the haunch. The value of $b = \frac{\text{min. I}}{\text{max. I}}$ is assumed to equal the ration of $\left(\frac{\text{min. d}}{\text{max. d}}\right)^3$, the beam width being considered constant and reinforcement disregarded."

These sentences suggest a good example of "double-talk." For, even if ratios could characterize beam shapes, if the ratios which a and b represent had been clearly stated and if the depth of a haunch could be an abstract proper fraction there would have been no good excuse for writing the sentences. It is the use of the "two ratios" that is important not their existence. Therefore, the value of these two sentences, by both tests, is zero or less.

*ACI Journal, Sept. 1944; *Proceedings* V. 41, p. 1.
†Consulting Engineer, Atlanta, Ga.

Second Example—"When blocks are placed and calculations for permissible loads are made as recommended, safe unit loads for spaced columns result (so far as strength or resistance to lateral buckling in a direction perpendicular to the mutually contacting surfaces of the blocks and individual members is concerned) and it is unnecessary to check the unstaid portions (from center of bolt in end blocks to center of spacer block) of the individual members".

The author of this sentence had something of value to tell but he failed so completely in telling it that the value of the sentence is zero. The writer is of the opinion that something like the following was intended:

If blocks are spaced as indicated in the schedule and bolted as shown in the sketch, the only calculation needed is to insure safety against bending edgewise of the timbers.

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The Effectiveness of Various Treatments and Coatings for Concrete in Reducing the Penetration of Kerosene*

By F. B. HORNIBROOK†
Member American Concrete Institute

SYNOPSIS

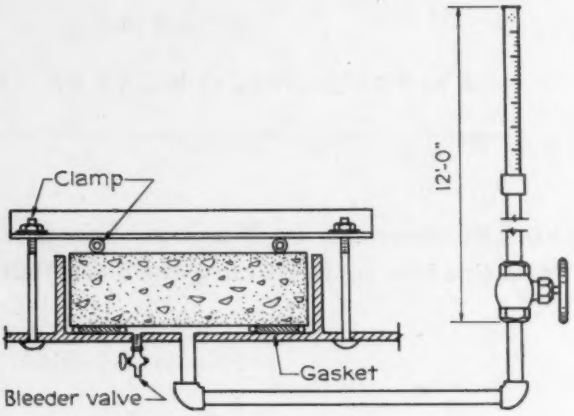
Measurements were made of the penetration of kerosene under a 12 foot pressure head into discs of concrete which had received various treatments or coatings. The tests were classified into 7 groups as follows: (1) reference concrete, (2) integral admixtures, (3) sodium silicate and magnesium fluosilicate treatments, (4) plaster coats, plain and with admixtures, (5) magnesium oxychloride type coatings, (6) linseed oil and spar varnish coatings, and (7) synthetic plastic and latex coatings. Comparisons of the relative rates of penetration of kerosene into the specimens of each group are given, together with a discussion of various other properties of each group.

INTRODUCTION

From tests reported in this JOURNAL in 1921‡ it was concluded that oils having viscosities lower than about 6 centipoises at 21C might leak excessively from unlined concrete tanks. Oils having viscosities lower than that value include Diesel oils, light fuel oils, kerosene and, of course, gasoline. The increased wartime use of concrete tanks for the storage of such oils prompted a limited investigation at the National Bureau of Standards of proposed materials for rendering the concrete adequately impermeable to light oils. Although there are many lining materials, methods of application, and observations of long-time effects that could not be included in a limited investigation, it is believed that some of the results obtained may be of value in connection with the problem of liquid storage in concrete.

*Received by the Institute, March 29, 1944.
†National Bureau of Standards, Washington, D. C.
‡"Further Tests of Concrete Tanks for Oil Storage," by George A. Smith. *Proc.* Am. Concrete Inst., Vol. 17, p. 22, 1932.

Fig. 1—Testing apparatus shown schematically



DESCRIPTION OF TESTS

The method adopted in the present tests consisted of treating or coating concrete discs and subjecting one face of the discs to a 12-ft head of kerosene which had a viscosity at 21-C of about 1.6 centipoises. The daily inflow of the kerosene was measured. The apparatus, shown schematically in Fig. 1, consisted essentially of a channel-iron base to which a 4-in. section of a 12-in. pipe was welded. An inflow pipe and a bleeder valve were threaded into the bottom of the base as illustrated. Difficulties experienced with leaking pipe joints were overcome with a quick-setting oil-resistant gasket cement. A graduated burette was sealed into the top of the standpipe for measurement of the inflow. The apparatus was set up in a constant temperature room.

The gasket for sealing the specimen in place consisted of two concentric rings spaced about 1½ in. apart, the inner ring of No. 12 copper wire and the outer ring solder wire; the space between the rings was filled with a hot glycerine glue gel* which is fluid when hot but tough and rubbery when cold. The specimen was put in place while the gasket material was hot, and the clamps tightened over the specimen.

The concrete used in fabricating the test specimens was a nominal 6-bag, 6½-gal. per bag concrete having a 2½ to 3-in. slump. In tests where integral admixtures were used, the mixing water was adjusted to obtain a slump of 3 in. After the molds were filled the concrete was vibrated for 10 seconds with an internal vibrator, then the surface was struck off and screeded. The specimens were disc-shaped, approximately 11½ in. in diameter and 4 in. deep. Most specimens were cast on an oiled plywood surface, but a few comparative specimens were cast on a lac-

*One part by weight of animal glue and 1 part of water are allowed to stand together over-night. One part of glycerine is then added and the mixture warmed in an oven to about 150 F. About 1 part by weight of fine sand (minus No. 50) is then added, and the mixture is ready for use. Gaskets can be re-used by simply rewarming.

quered plywood surface. The cast face of the specimen was used as the test surface.

General practice was to cure the specimens one day in the mold, 6 days in the damp room, and 7 days in the laboratory air. Specimens that were to receive paint-type coatings were usually given a "sacking" with a 1:1 cement-fine sand (minus No. 50) grout, upon removal from the molds, to fill small surface voids and then, after the 7-day drying period, were given the first application of the prescribed coating. Specimens that were to receive plaster-coats of mortar were, upon removal from the forms, roughened with a steel brush, given a thin grout coat followed by the plaster coat and then the usual curing. The ages of the specimens at the time of testing are listed in Table 1.

The amount of fluid inflow was determined daily. For those specimens having an appreciable inflow, the amount was usually large at first and then diminished gradually until an approximately constant inflow was reached in usually 8 to 12 days. When the inflow became reasonably constant the average daily amount for 3 to 5 days was reported as the inflow for the specimen. All tests were made in duplicate, unless otherwise noted. For specimens having appreciable inflows (greater than 0.4 gal. per 1000 sq. ft. per 24 hrs.) the average variation from the mean value for duplicate specimens was 14 per cent.

Many tests which are not reported were made on materials submitted by manufacturers and others interested in the development of satisfactory linings for the storage of gasoline. For various reasons these materials were unsatisfactory, the most frequent cause being their instability to water, as manifested by visible softening or disintegration of the coating after storage in the fog room for two weeks. Tests of other materials are reported, however, even though these materials were found unsatisfactory because they represent commonly suggested approaches to the solution of the problem at hand.

DISCUSSION OF RESULTS

The results obtained are presented in Table 1. Each value listed is the average of two specimens unless special notation is made. The retests listed in the fourth column were, as a rule, made after the specimens had dried in the laboratory 6 to 8 weeks after completion of the initial tests. The tests have been divided into 7 groups, as follows: (1) reference concrete; (2) integral admixtures; (3) sodium silicate and magnesium fluosilicate treatments; (4) plaster coats, plain and with admixtures; (5) magnesium oxychloride-type coatings; (6) linseed oil and spar varnish coatings; and (7) synthetic plastic and latex coatings.

TABLE 1—RATE OF PENETRATION OF KEROSENE UNDER A 12-FT HEAD INTO PLAIN AND SPECIALLY TREATED CONCRETE SPECIMENS

Spec. No.	Description and Treatment of Specimen	Approximate Age at Intial Test Days	Liquid Inflow gal. per 1000 sq. ft. per 24 hours: Initial Test	Liquid Inflow gal. per 1000 sq. ft. per 24 hours: Retest After Drying 6 to 8 Weeks	Remarks
	Group 1—Reference Concrete				
88	6 bag, 6.5 gal/bag concrete	14	1.7	11.3	Cast in lacquered forms
89	6 bag, 6.5 gal/bag concrete	14	2.2	17.1	Cast in oiled forms
115a	Field prepared Gunite specimen	unknown	117 [a]	....	Specimen had probbably dried several weeks before test
	Group II—Integral Admixtures				
94	Admix. A, inorganic, contained chloride, sulfate, soluble and insoluble silica. Added in ratio of 3½ lb/bag of cement	14	1.0	9.1	Cast in lacquered forms
95	Admix. A, same as above	14	2.6	11.0	Cast in oiled forms
96	Admix. B, contained sulfonated lignen calcium chloride and a filler, added in ratio of 1 lb/bag of cement, mixing water reduced by 11% to keep slump constant	14	1.3	7.6	Lacquered forms
97	Admix. C, contained a carbohydrate and a filler, added in ratio of 1 lb/bag of cement, mixing water reduced 6%	14	1.7	6.5	Lacquered forms
	Group III—Silicate and Fluosilicate Treatments				
113	Plain concrete—troweled face	14	1.9	....	Test made on troweled face
98	High silica sodium silicate on troweled face, 2 coats, 1:4 by volume	21	2.1	6.1	Test made on troweled face
101	Same as 98 but applied on cast face	21	4.0	12.4	Tested on cast face
102	Mag. fluosilicate on troweled face, 2 coats, first dil. 1:16, second dil. 1:4 by weight	21	1.9	....	Tested on troweled face
103	Same as 102 but applied on cast face	21	2.8	....	Tested on cast face
43	Commercial sodium silicate Paint D containing iron oxide pigment	21	0.2	....	Coating unstable in water

Table 1—Continued

Spec. No.	Description and Treatment of Specimen	Approximate Age at Intial Test Davs	Liquid Inflow gal. per 1000 sq. ft. per 24 hours		Remarks
			Initial Test	Retest After Drying 6 to 8 Weeks	
	Group IV—Mortar Coats				
104	¼ in. troweled mortar coat 1:2:5 mortar troweled hard	14	0.1	1.5	
105	Same as 104 with 2 coats high silica sodium silicate diluted 1:4 applied	21	0.3	2.5	
54	Powdered iron—type Admix. E in mortar 15% admix. in 1:2½ mortar	14	9.9[b]	....	
56	Powered iron—agent Admix. F in mortar. 5 successive layers as directed by manufacturer	21	17.	....	
42	Caustic liquid type Admix. G in mortar approx. 33% sol. calc. as K_2CO_3	21	excessive	....	Liquid flowed laterally through mortar
	Group V—Magnesium Oxychloride Type Coatings				
17	Magnesium oxychloride type paint H	21	1.6	....	Fairly stable to moisture
19	Magnesium oxychloride type Paste I; ¼" thick coat with cloth mesh	14	3.6[a]	....	
19a	Magnesium oxychloride type Paste I; neat, ⅛" thick	14	.8[a]	....	
111	Magnesium oxychloride type Paste I	14	10.	....	Retest on new sample Surface crazed
38	Magnesium oxychloride type Paste J. Coating about ⅛" thick	14	0.1	43.[c]	
52	Same as 38 but preceded by a powdered iron-cement grout	14	<.05	....	
52a	Same as 52 but damp-cured (fog room)	14	6.6	....	Coating developed gel-like spots
	Group VI—Linseed Oil and Varnish				
64	Linseed oil; 2 coats, thinned, dried 6 weeks	50	2.4	....	
51	One coat linseed oil plus one coat spar varnish, each dried 1 week	28	.2	....	
57	Spar varnish; 3 coats, dried 2 weeks altogether	28	.2	....	

Table 1 concluded next page

Table 1 concluded

Spec. No.	Description and Treatment of Specimen	Approximate Age at Intial Test Days	Liquid Inflow gal. per 1000 sq. ft. per 24 hours: Initial Test	Liquid Inflow gal. per 1000 sq. ft. per 24 hours: Retest After Drying 6 to 8 Weeks	Remarks
	Group VII—Synthetic Plastics and Latex				
59	Polyvinyl Chloride Type Paint K; 3 coat application	21	.4	3.0[d]	
73	Polyvinyl Chloride Type Paint L; 3 coat application	30	<.05	....	
107	Emulsified polysulfide latex type Paint M, 2 primer coats, one finish coat	14	<.05	....	
115	Paint M, job prepared specimens on Gunite concrete. Fabric reinforcement	....	<.05	[e]	

[a] Value for 1 specimen only.
[b] Large difference between duplicates; 54a = 4.5 gal., 54b = 15.3 gal.
[c] Retest made approximately 1 year after initial test. Specimens were drying in laboratory during that time.
[d] Retest at 14 months. Specimen stored in laboratory.

The reference concretes had comparatively low penetration for the initial test (concrete aged 14 days) but during the retest, after the concrete had dried, the penetration was high. This large increase of penetration as the concrete dries illustrates one of the probable advantages of placing concrete tanks under ground where drying may not be severe.

The use of the integral admixtures of group 2 indicates little advantage in the initial tests. After drying, however, the penetration was somewhat reduced with respect to the dried reference concrete. A penetration of 1 gal per 1000 sq ft per 24 hours is roughly equivalent to a one percent annual loss from a 50,000 or a 100,000 gal. tank, assuming uniform loss from the sides and the bottom. It is indicated therefore that the admixtures do not sufficiently reduce the penetration to warrant their use for the purpose of rendering concrete impermeable to oils as light as kerosene.

Coupled with the tests of group 1 and group 2 is a comparison of concretes cast on oiled plywood and on plywood which had been coated with acetate type commercial form lacquer. In both the initial tests and in the retests, the specimens cast on the lacquered surface had lower rates of penetration. The reason for this reduction is not apparent, unless the lacquered forms act in a minor way as absorptive forms.

The tests in group 3 indicate that sodium silicate and magnesium fluosilicate, as applied, did not confer much benefit, though the application to the trowelled face was considered somewhat more effective than application to the cast face. The commercial pigmented sodium silicate paint was very effective in reducing penetration, but this coating proved to be highly unstable in a saturated atmosphere or in water.

The group 4 tests on mortar coat applications indicated some success with a 1:2½ mortar applied ⅛ to ¼-in. thick when the specimens were removed from the mold at about 24 hours. The mortar was trowelled hard when it was 3 to 4 hours old. This test is probably not representative of what might occur on a job, since a small specimen is apt to receive more favorable troweling. Furthermore, shrinkage cracks probably did not develop on the small specimens as extensively as they might on large surfaces. The mortars containing iron-type admixtures E and F were applied in accordance with manufacturer's directions for waterproofing concrete. The mortar containing admixture G was placed according to directions for coating concrete tanks "for storing mineral oils (crude oil, fuel oil)". None of the mortar coats containing admixtures were effective in stopping the inflow of kerosene.

The magnesium oxychloride-type paste coatings tested in group 5 have been suggested as possible linings for gasoline storage. The cement (I, Table 1) had a tendency to craze and was relatively ineffective. This lining appeared to be unaffected by storage in water for 2 weeks. The cement (J) had a low rate of penetration on the initial test, except for the specimen kept in the fog-room after application of coating. The application, first, of a powdered iron grout coat on specimen No. 52 appeared to be beneficial, but further testing would be required to determine if the benefit is real or only accidental. After standing in the laboratory about 1 year, specimens coated with cement (J) showed excessive crazing. A very small concrete tank lined with cement (J) was buried in the ground so the top of the tank was even with the soil surface. No protection from the weather was provided. The tank was filled with kerosene and a cover sealed on. The leakage from the tank has been low, but not measurable due to the "creeping" of the kerosene over the top of the tank. After 16 months there is no appearance of crazing or of swelling of the lining.

Materials of this type have the advantage of low materials cost, low cost of application and simplicity of application. The limited laboratory tests available indicate their use might be satisfactory for light fuel and diesel oil storage where drying conditions are not severe. Job applications have been made and it is hoped that service reports will be made available.

Considering the length of drying given the linseed oil applications of group 6, it appears that a moderate decrease in penetration was effected with this material. The principal question both with the linseed oil and spar varnish applications, is the lasting power of these materials under field exposure conditions.

The materials in group 7 are representative of the type of linings that have come into use for aviation gasoline storage since the inception of this study. Their materials costs and application costs are high, however, leaving considerable room yet for the development of low-cost but satisfactory lining materials.

ACKNOWLEDGEMENT

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Two-Way Reinforced Concrete Slabs*

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Member American Concrete Institute

SYNOPSIS

Design formulas for two-way reinforced concrete slabs are given in the ACI Building Regulations (318-41). No derivations, however, are included for these formulas. While outstanding scientific works are at hand dealing with this problem (Timoshenko, Westergaard, etc.), it is obvious that they are out of reach for the average office designer. The foregoing simplified method, developed by Dr. Henri Marcus, may clarify certain principles and help visualize the effect of the torsional moments.

As a first approach in analyzing the bending moments in slabs supported on four edges, a typical slab was divided in strips of unit width in both directions, and the strips were treated as independent beams. At the intersection of the strips from two directions, both beams have equal deflections and moments of inertia, which led to a simple way of determining the portions of load to be carried by these beams.

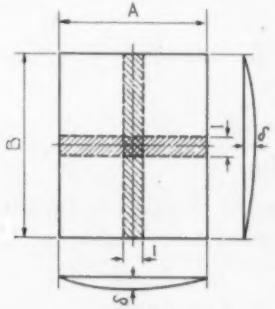

Fig. 1

These assumptions, however, did not take into account the lateral restraining forces which act upon these imaginary strips, and which tend to decrease the deflections. As a result, the obtained bending moments

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did not correspond to the actual ones. To overcome these difficulties and to provide for an economical design, certain codes (as the "Conseil général des ponts et chaussées" in France) permitted the use of some empirical coefficients, which decreased the bending moments by 10 to 30 per cent. Naturally, this method was very crude because the actual decrements do not follow a straight line proportion, and are also dependent on the restraint and continuity along the edges.

Meanwhile several European and American scientists developed and perfected the "exact" theory of plates and slabs. But to understand and apply this theory in practice requires a very good knowledge of the theory of elasticity and of higher mathematics, not considering the time to be spent in arriving at a solution.

Thus there were the two alternates: (1) The "exact" theory, known and applied by the very few, and (2) the "approximate" theory which was simply incorrect. Dr. Henri Marcus, in his remarkable book "Vereinfachte Berechnung biegsamer Platten" has bridged the difficulties in a unique way. Comparing the bending moments obtained by the "*exact*" theory with those obtained by dividing the slab in strips in two directions, he derived an easy formula for the practically precise bending moment

$$M_A = m_A (1 - \varphi_A) = m_A \nu_A \qquad M_B = m_B (1 - \varphi_B) = m_B \nu_B \ldots\ldots (1)$$

where M_A indicates the final Moment in "A" direction

M_B indicates the final Moment in "B" direction.

m_A indicates the Moment obtained by loading the strip in "A" direction with w_A.

m_B indicates the Moment obtained by loading the strip in "B" direction with w_B.

($w_A + w_B = w$ = total uniformly distributed load per unit square area), while

$$\varphi_A = \frac{5}{6} \times \frac{A^2}{B^2} \times \frac{m_A max}{M_{oA}} \qquad \varphi_B = \frac{5}{6} \times \frac{B^2}{A^2} \times \frac{m_B max}{M_{oB}} \ldots\ldots\ldots (2)$$

M_{oA} and M_{oB} indicating the respective Bending Moments of strips of unit width, simply supported and loaded with the *full* load of w per linear ft.

The derivations for the Equations (2) are beyond the scope of this article and will not be given here. They, however, satisfy the requirements for a correct coefficient to supply the "*exact*" bending moments. It could be proven that there is practically no discrepancy between moments obtained by Equations (1) and (2), and those obtained by theory of elasticity. It should here be mentioned, however, that the torsional moments, which produce the decrement of the bending moments, can not be computed easily by approximate methods. But there is very little

need, if any, for these torsional moments, as far as structural design is concerned.

In this rather popular article, the subject of two-day slabs will be confined to the derivations of bending moments for slabs of different edge conditions, and the graphical illustration of the torsional moments. (The curved lines of the torsional moments are replaced by straight lines for better illustration.)

Case a. Slab freely supported on all four edges.

w = Load per square foot of slab (D.L.+L.L.).

w_A and w_B = portions of w in directions A and B respectively ($w_A + w_B = w$).

A and B = Spans in directions A and B respectively.

Maximum deflection of one foot wide middle-strip in A direction:

$$\frac{5}{32} \times \frac{w_A \times A^4}{E \times t_1^3}$$

Maximum deflection of one foot wide middle-strip in B direction:

$$\frac{5}{32} \times \frac{w_B \times B^4}{E \times t_1^3}$$

The maximum deflection occurs at the middle-span, where both deflections are equal, or

$w_A \times A^4 = w_B \times B^4$, and thus

$$w_A = \frac{w \times B^4}{A^4 + B^4} \text{ and } w_B = \frac{w \times A^4}{A^4 + B^4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$m_A \max = \frac{w \times A^2}{8} \frac{B^4}{A^4 + B^4}; \; M_{oA} = \frac{w \times A^2}{8},$$

$$\text{and } \frac{m_A \, max}{M_{oA}} = \frac{B^4}{A^4 + B^4}; \quad m_B \, max = \frac{w \times B^2}{8} \frac{A^4}{A^4 + B^4};$$

$$M_{oB} = \frac{w \times B^2}{8}, \text{ and } \frac{m_B \, max}{M_{oB}} = \frac{A^4}{A^4 + B^4}$$

$$\varphi_A = \varphi_B = \varphi = \frac{5}{6} \times \frac{A^2 \times B^2}{A^4 + B^4}; \text{ or } (1 - \varphi) = \nu_a =$$

$$1 - \left(\frac{5}{6} \times \frac{A^2 \times B^2}{A^4 + B^4}\right), \text{ and finally}$$

$$M_A \, max = \frac{1}{8} \times w_A \times A^2 \times \nu_a; \text{ and } M_B \, max = \frac{1}{8} \times w_B \times B^2 \times \nu_a; \quad (4)$$

It is an easy task to set up tables for the values of w_A, w_B, ν_a. For a square plate, the value of ν_a is equal to 0.583 which certainly is a substantial reduction of Moment-value.

$$\text{Maximum torsional moment } T_a = \pm \frac{w \times \nu_a}{6} \times \frac{A^3 \times B^3}{A^4 + B^4} \quad \ldots\ldots (5)$$

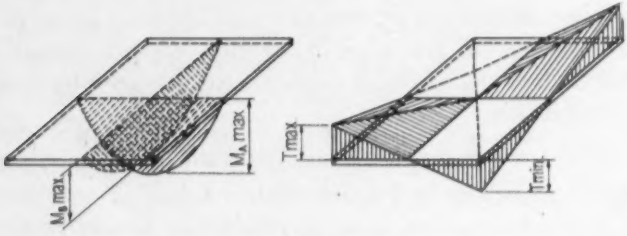


Fig. 2

Case b. Slab fixed along all four edges.

Maximum deflections in A and B directions: $\frac{1}{32} \times \frac{w_A \times A^4}{E \times t_1^3}$ and

$$\frac{1}{32} \times \frac{w_B \times B^4}{E \times t_1^3}, \text{ which gives } w_A = \frac{w \times B^4}{A^4 + B^4} \text{ and } w_B = \frac{w \times A^4}{A^4 + B^4}; \quad (6)$$

$$m_A\ max = \frac{w_A \times A^2}{24}; \quad m_B\ max = \frac{w_B \times B^2}{24}; \quad M_{oA} = \frac{w \times A^2}{8};$$

$$M_{oB} = \frac{w \times B^2}{8}; \quad \varphi_b = \frac{5}{18} \times \frac{A^2 \times B^2}{A^4 + B^4}; \quad \nu_b = (1 - \varphi_b)$$

$$M_A\ max = \frac{1}{24}\ w_A \times A^2 \times \nu_b; \quad M_B\ max = \frac{1}{24}\ w_B \times B^2 \times \nu_b \quad \ldots (7)$$

For a square plate, the value of $\nu_b = 0.861$, which is a far smaller reduction than that for the simply supported slab. The 1/24 coefficient is theoretically correct, but it is unusual to apply more than 1/16 or 1/18 for practical purposes.

Maximum torsional moment

$$T_b = \pm \frac{3}{100} \times \frac{w \times A^3 \times B^3}{A^4 + B^4} \times \nu_b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

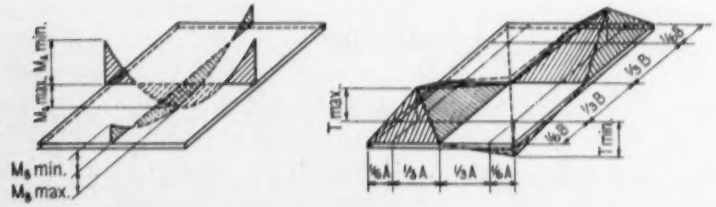


Fig. 3

Case c. Slab freely supported along three sides, and fixed on fourth side.

$m_A\ max = \frac{9}{128} \times w_A \times A^2$; $m_A\ min. = -\frac{1}{8} \times w_A \times A^2$;

Deflection at middlespan: $\frac{2}{32} \times w_A \times \frac{A^4}{E \times t_1^3}$

$m_B\ max = \frac{1}{8} \times w_B \times B^2$, while the deflection at middlespan:

$\frac{5}{32} \times w_B \times \frac{B^4}{E \times t_1^3}$, thus $w_A = \frac{w \times 5B^4}{2A^4 + 5B^4}$; $w_B = \frac{w \times 2A^4}{2A^4 + 5B^4}$; (9)

and $m_A\ max = \frac{9}{128} \times A^2 \times \frac{w \times 5B^4}{2A^4 + 5B^4}$ also:

$m_B\ max = \frac{1}{8} \times B^2 \times \frac{w \times 2A^4}{2A^4 + 5B^4}$;

$M_{oA} = \frac{1}{8} \times w \times A^2$; $M_{oB} = \frac{1}{8} \times w \times B^2$

According to Equations No. 2:

$\varphi_{cA} = \frac{5}{6} \times \frac{9 \times 8}{128} \times \frac{5A^2 \times B^2}{2A^4 + 5B^4} = \frac{75}{32} \times \frac{A^2 \times B^2}{2A^4 + 5B^4}$

and $\nu_{cA} = (1 - \varphi_{cA}) = 1 - \left(\frac{75}{32} \times \frac{A^2 \times B^2}{2A^4 + 5B^4}\right)$

$\varphi_{cB} = \frac{5}{6} \times \frac{2 \times A^2 \times B^2}{2A^4 + 5B^4} = \frac{5}{3} \times \frac{A^2 \times B^2}{2A^4 + 5B^4}$

and $\nu_{cB} = (1 - \varphi_{cB}) = 1 - \left(\frac{5}{3} \times \frac{A^2 \times B^2}{2A^4 + 5B^4}\right)$

$M_A\ max = \frac{9}{128} \times w_A \times A^2 \times \nu_{cA}$; $M_B\ max = \frac{1}{8} \times w_B \times B^2 \times \nu_{cB}$;

M fixed side: $-\frac{1}{8} \times w_A \times A^2$; ..... (10)

Torsional moment $T_c = \frac{w \times A^3 \times B^3}{2A^4 + 5B^4} \times (.0985 + .2606\ \nu_c)$

Torsional moment $T'_c = \frac{w \times A^3 \times B^3}{2A^4 + 5B^4} \times (.0803 + .2133\ \nu'_c)$ .. (11)

Case d. Slab freely supported on two opposite sides in one direction and fixed on two opposite sides in the other direction.

$m_A\ max = \frac{1}{24} \times w_A \times A^2$, while the deflection in middlespan:

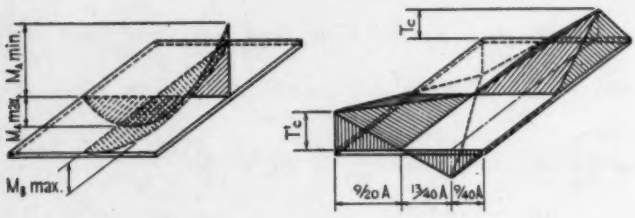


Fig. 4

$$\frac{1}{32} \times w_A \times \frac{A^4}{E \times t_1^3}$$

$m_B\ max = \frac{1}{8} \times w_B \times B^2$, while the deflection in middlespan:

$$\frac{5}{32} \times w_B \times \frac{B^4}{E \times t_1^3}$$

The two deflections being equal, we get:

$$w_A = \frac{w \times 5B^4}{A^4 + 5B^4}, \text{ and } w_B = \frac{w \times A^4}{A^4 + 5B^4} \quad \ldots\ldots (12)$$

Again $\quad M_{oA} = \frac{1}{8} \times w \times A^2; \qquad M_{oB} = \frac{1}{8} \times w \times B^2;$

$$\varphi_{dA} = \frac{5}{6} \times \frac{8 \times w_A \times A^2}{24 \times w \times B^2} = \frac{25}{18} \times \frac{A^2 \times B^2}{A^4 + 5B^4},$$

$$\text{and } \nu_{dA} = (1 - \varphi_{dA}) = 1 - \left(\frac{25}{18} \times \frac{A^2 \times B^2}{A^4 + 5B^4}\right)$$

$$\varphi_{dB} = \frac{5}{6} \times \frac{8 \times w_B \times B^2}{8 \times w \times A^2} = \frac{5}{6} \times \frac{A^2 \times B^2}{A^4 + 5B^4},$$

$$\text{and } \nu_{dB} = (1 - \varphi_{dB}) = 1 - \left(\frac{5}{6} \times \frac{A^2 \times B^2}{A^4 + 5B^4}\right)$$

$$M_A\ max = \frac{1}{24} \times w_A \times A^2 \times \nu_{dA};\ M_B\ max: \frac{1}{8} \times w_B \times B^2 \times \nu_{dB};$$

$$M \text{ fixed sides: } -\frac{1}{12} \times w_A \times A^2 \quad \ldots\ldots (13)$$

Maximum torsional moment

$$T_d = \frac{w \times \nu_{dB}}{6} \times \frac{A^3 \times B^3}{A^4 + 5B^4} \quad \ldots\ldots (14)$$

Case e. Slab freely supported along two meeting sides, and fixed on two opposite sides.

$m_A\ max: \frac{9}{128} \times w_A \times A^2$, while the deflection in middlespan:

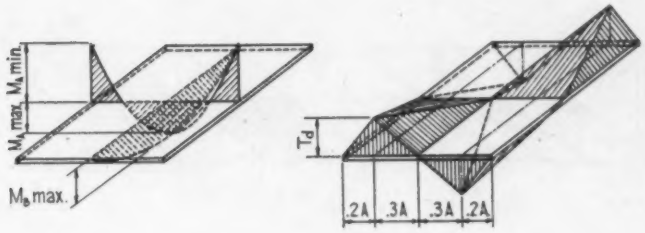


Fig. 5

$$\frac{2}{32} \times \frac{w_A \times A^4}{E \times t_1^3};$$

m_B *max:* $\frac{9}{128} \times w_B \times B^2$, while the deflection in middlespan:

$$\frac{2}{32} \times \frac{w_B \times B^4}{E \times t_1^3};$$

$$w_A = \frac{w \times B^4}{A^4 + B^4}; \qquad w_B = \frac{w \times A^4}{A^4 + B^4}; \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

$$\varphi_{eA} = \varphi_{eB} = \frac{5}{6} \times \frac{9 \times 8}{128} \times \frac{w_A}{w} \times \frac{A^2}{B^2} =$$

$$\frac{5}{6} \times \frac{9 \times 8}{128} \times \frac{w_B}{w} \times \frac{B^2}{A^2} = \frac{15}{32} \times \frac{A^2 \times B^2}{A^4 + B^4}, \text{ and}$$

$$\nu_e = (1 - \varphi_e) = 1 - \left(\frac{15}{32} \times \frac{A^2 \times B^2}{A^4 + B^4}\right)$$

Thus the final positive and negative moments:

$$M_A\ max: \frac{9}{128} \times w_A \times A^2 \times \nu_e; \quad M_B\ max: \frac{9}{128} \times w_B \times B^2 \times \nu_e;$$

$$M_A\ F.E. = -\frac{1}{8} \times w_A \times A^2; \qquad M_B\ F.E. = -\frac{1}{8} \times w_B \times B^2. \quad (16)$$

Torsional moments:

$$T_e = \frac{5}{297} \times \frac{A^3 \times B^3}{A^4 + B^4} \times w \times (1.064 + 2.815\ \nu_e)$$

$$T'_e = \frac{5}{263} \times \frac{A^3 \times B^3}{A^4 + B^4} \times w \times (1.064 + 2.815\ \nu_e)$$

$$T''_e = \frac{5}{243} \times \frac{A^3 \times B^3}{A^4 + B^4} \times w \times (1.064 + 2.815\ \nu_e) \ldots\ldots\ldots\ldots (17)$$

Case f. Slab freely supported along one side only and fixed on all the other three sides.

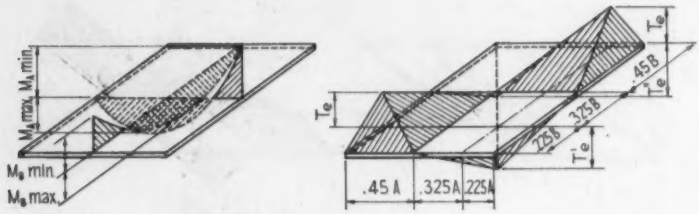


Fig. 6

$m_A\ max$: $\frac{1}{24} \times w_A \times A^2$; while the deflection in middlespan:

$\frac{1}{32} \times \frac{w_A \times A^3}{E \times t_1^3}$; $\qquad m_B\ max$: $\frac{9}{128} \times w_B \times B^2$;

while the deflection in middlespan: $\frac{2}{32} \times \frac{w_B \times B^4}{E \times t_1^3}$

$$w_A = \frac{w \times 2B^4}{A^4 + 2B^4} \qquad w_B = \frac{w \times A^4}{A^4 + 2B^4}; \quad \ldots\ldots\ldots\ldots (18)$$

$$\varphi_{fA} = \frac{5}{6} \times \frac{8}{24} \times \frac{w_A}{w} \times \frac{A^2}{B^2} = \frac{5}{9} \times \frac{A^2 \times B^2}{A^4 + 2B^4};$$

$$\nu_{fA} = (1 - \varphi_{fA}) = 1 - \left(\frac{5}{9} \times \frac{A^2 \times B^2}{A^4 + 2B^4}\right);$$

$$\varphi_{fB} = \frac{5}{6} \times \frac{9 \times 8}{128} \times \frac{w_B}{w} \times \frac{B^2}{A^2} = \frac{15}{32} \times \frac{A^2 \times B^2}{A^4 + 2B^4};$$

$$\nu_{fB} = (1 - \varphi_{fB}) = 1 - \left(\frac{15}{32} \times \frac{A^2 \times B^2}{A^4 + 2B^4}\right);$$

Final positive and negative moments:

$M_A\ max$: $\frac{1}{24} \times w_A \times A^2 \times \nu_{fA}$;

$M_B\ max$: $\frac{9}{128} \times w_B \times B^2 \times \nu_{fB}$;

$M_A\ F.E. = -\frac{1}{12} \times w_A \times A^2$;

$$M_B\ F.E. = -\frac{1}{8} \times w_B \times B^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Torsional moments: $\mathrm{T}_f = \frac{5}{36} \times \frac{A^3 \times B^3}{A^4 + 2B^4} \times \frac{w \times \nu_{fA}}{1 + \nu_{fA}}$

$$\mathrm{T}_f' = \frac{5}{44} \times \frac{A^3 \times B^3}{A^4 + 2B^4} \times \frac{w \times \nu_{fA}}{1 + \nu_{fA}} \quad \ldots\ldots (20)$$

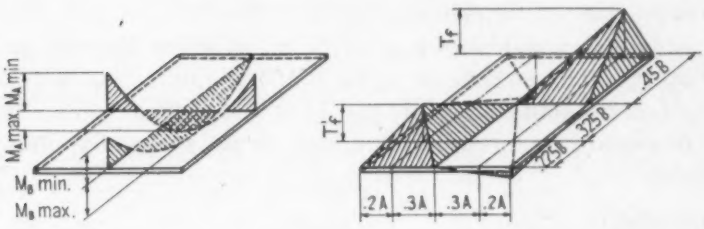


Fig. 7

Cases *a* to *f* include all possible conditions for individual two-way reinforced concrete slabs. For the practical application of the derived formulas Tables 1_A, 1_B, and 1_C—give simplified results by using a single coefficient n_A or n_B to obtain the maximum bending moments.

In actual practice continuous slabs over several bays in both directions are frequently used. To obtain maximum moments in continuous slabs, variations of the so called checker-plate loading shall be applied. (Fig. 8.)

An equivalent state of stress can be produced,

(a) by loading simultaneously all panels with $p_1 = \frac{1}{2}$ L.L. and thereafter,

(b) by loading alternatively,

the hatched areas with $p_2 = \frac{1}{2}$ L.L.

the unhatched areas with $p_2 = -\frac{1}{2}$ L.L.

For the conditions (a) the inside edges of each panel may be considered as fixed, whereas for the conditions (b) all four edges of each panel are acting as freely supported.

This schedule enables us to determine the greatest moments in each bay independently.

For practical illustration let us consider a slab, continuous over three bays in both directions, columns being spaced twenty and eighteen feet apart respectively. The slab is loaded with 100 lb. per sq.ft. live load applied alternately as shown on the loading schemes *x*, *y*, and *z*.

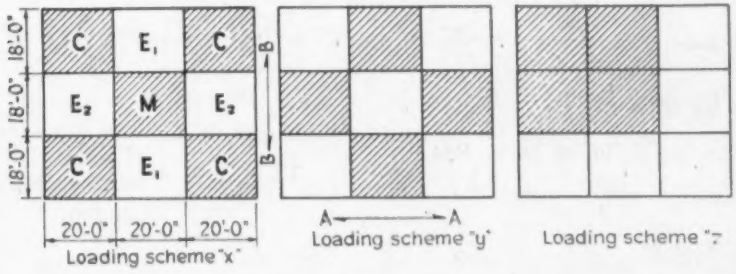


Fig. 8

The outer edges of the slab are considered freely supported.

Let us assume a slab thickness of 5 in. weighing approximately 60 lb. per sq. ft. Thus we have $60 + \frac{1}{2} \times 100 = 110$ lb. per sq. ft. downward load on the actual structure and $\frac{1}{2} \times 100 = 50$ lb. per sq. ft. alternately downward or upward acting load on an assumedly simply supported slab.

Corner Slab C.

(a) Maximum positive moment at panel center for loading scheme x.

Apply Case e for the 110 lb. per sq. ft. load,
and Case a for the 50 lb. per sq. ft. load.
B/A = 18/20 = .90 and by using Tables 1_C and 1_A.

$$M_A\ max. = \frac{110 \times 20^2}{46.58} \pm \frac{50 \times 20^2}{34.26} \ldots\ldots\ldots\ldots\ldots\ldots 1.53 \text{ ft. kips.}$$

$$M_B\ max. = \frac{110 \times 18^2}{30.56} \pm \frac{50 \times 18^2}{22.36} \ldots\ldots\ldots\ldots\ldots\ldots 1.89 \text{ ft. kips.}$$

(b) Maximum negative moment at the supports for loading scheme z.

$w_A = (60 + 100) \times .40 = 64$ lb/sq. ft.
$w_B = (60 + 100) \times .60 = 96$ lb/sq. ft.

M_A min. $= -1/8 \times 64 \times 20^2 \times .85$ (at face of beam) $= -2.72$ ft. kips

M_B min. $= -1/8 \times 96 \times 18^2 \times .85$ (at face of beam) $= -3.30$ ft. kips

Using concrete of f'_c:3000 psi, f_c:1250 psi, n:10, and reinf. steel of f_s:20,000 psi.

$$A_{sApos.} = \frac{1.53}{1.44 \times 3.5} = .31 \text{ sq. in.} \quad \text{Use: } \tfrac{1}{2}'' \phi \text{ Bars} - 7\tfrac{1}{2}'' \text{ o.c.}$$

$$A_{sBpos.} = \frac{1.89}{1.44 \times 4} = .33 \text{ sq. in.} \quad \text{Use: } \tfrac{1}{2}'' \phi \text{ Bars—7}'' \text{ o.c.}$$

$$A_{sAneg.} = \frac{2.72}{1.44 \times 4} = .47 \text{ sq. in.} \quad \text{Use: } \tfrac{5}{8}'' \phi \text{ Bars—}7\tfrac{1}{2}'' \text{ o.c.}$$

$$A_{sBneg.} = \frac{3.30}{1.44 \times 4} = .57 \text{ sq. in.} \quad \text{Use: } \tfrac{5}{8}'' \phi \text{ Bars—}6\tfrac{1}{2}'' \text{ o.c.}$$

Check for shear:

$$V = \tfrac{1}{2} \times 96 \times 18 = 864 \text{ lb.}; \quad v = \frac{864 \times 8}{12 \times 7 \times 4} = 20.6 \text{ psi. no bent-up bars are needed.}$$

End Slab E_1.

(a) Maximum positive moment for loading scheme y;

Apply Case *f* for the 110 lb. per sq. ft. load.
and Case *a* for the 50 lb. per sq. ft. load.

$$M_A\ max. = \frac{110 \times 20^2}{52.51} \pm \frac{50 \times 20^2}{34.26} = \quad 1.42 \text{ ft. kips.}$$

$$M_B\ max. = \frac{110 \times 18}{39.35} + \frac{50 \times 18}{22.36} = \quad 1.63 \text{ ft. kips.}$$

(b) Maximum negative moments at the supports for loading scheme *z*.

$w_A = (60 + 100) \times .568 = \quad 91$ lb/sq. ft.
$w_B = (60 + 100) \times .432 = \quad 69$ lb/sq. ft.

M_A min $= -1/11 \times 91 \times 20 \times .85$ (at face of beam) $=$
-2.81 ft. kips

M_B min. $= -1/8 \times 69 \times 18 \times .85$ (at face of beam) $=$
-2.38 ft. kips

$$A_{sApos.} = \frac{1.42}{1.44 \times 3.5} = .29 \text{ sq. in.}$$ Use: ½″ ϕ Bars − 8″ o.c.

$$A_{sBpos.} = \frac{1.63}{1.44 \times 4} = .29 \text{ sq. in.}$$ Use: ½″ ϕ Bars − 8″ o.c.

$$A_{sAneg.} = \frac{2.81}{1.44 \times 4} = .49 \text{ sq. in.}$$ Use: ⅝″ ϕ Bars − 7½″ o.c. (from C)

$$A_{sBneg.} = \frac{2.38}{1.44 \times 4} = .42 \text{ sq. in.}$$ Use: ⅝″ ϕ Bars − 8½″ o.c.

End Slab E_2.

(a) Maximum positive moment for loading scheme *y*
Apply Case *f*. for the 110 lb. per sq. ft. load
and Case *a* for the 50 lb. per sq. ft. load

$$M_A\ max. = \frac{110 \times 20^2}{65.30} \pm \frac{50 \times 20^2}{34.26} = \quad 1.26 \text{ ft. kips}$$

$$M_B\ max. = \frac{110 \times 18^2}{38.84} \pm \frac{50 \times 18^2}{22.36} = \quad 1.65 \text{ ft. kips}$$

(b) Maximum negative moments at the supports for loading scheme **z**.

$w_A = (60 + 100) \times .427 = \quad 40$ lb/sq. ft.
$w_B = (60 + 100) \times .753 = \quad 120$ lb./sq. ft.

M_A min. $= -1/8 \times 40 \times 20^2 \times .85 = \quad -1.70$ ft. kips
M_B min. $= -1/11 \times 120 \times 18^2 \times .85 = \quad -3.00$ ft. kips

$$A_{sApos.} = \frac{1.26}{1.44 \times 3.5} = .25 \text{ sq. in.}$$ Use: ½″ ϕ Bars − 9½″ o.c.

$$A_{sBpos.} = \frac{1.65}{1.44 \times 4} = .29 \text{ sq. in.}$$ Use: ½″ ϕ Bars − 8″ o.c.

$$A_{sAneg.} = \frac{1.70}{1.44 \times 4} = .30 \text{ sq. in.}$$ Use: $\frac{5}{8}''$ ϕ Bars — 11″ o.c.

$$A_{sBneg.} = \frac{3.00}{1.44 \times 4} = .52 \text{ sq. in.}$$ Use: $\frac{5}{8}''$ ϕ Bars — $6\frac{1}{2}''$ o.c. (from C)

Middle Slab M.

Maximum positive moment produced by loading scheme x;
Maximum negative moment produced by loading scheme z.
Apply Case b. for the 110 lb. per sq. ft. load, and Case a. for the 50 lb. per sq. ft. load.

$$M_{Amax.} = \frac{110 \times 20^2}{70.10} + \frac{50 \times 20^2}{34.26} = 1.22 \text{ Ft. Kips}$$

$$M_{Bmax.} = \frac{110 \times 18^2}{46.00} + \frac{50 \times 18^2}{22.36} = 1.50 \text{ Ft. Kips.}$$

$$M_{Amin.} = 1/11 \times 64 \times 20^2 \times .85 = -2.00 \text{ Ft. Kips.}$$

$$M_{Bmin.} = 1/11 \times 96 \times 18^2 \times .85 = -2.40 \text{ Ft. Kips.}$$

$$A_{sApos.} = \frac{1.22}{1.44 \times 3.5} = .25 \text{ sq. in.}$$ Use: $\frac{1}{2}''$ ϕ Bars — $9\frac{1}{2}''$ o.c.

$$A_{sBpos.} = \frac{1.50}{1.44 \times 4} = .26 \text{ sq. in.}$$ Use: $\frac{1}{2}''$ ϕ Bars — 9″ o.c.

$$A_{sAneg.} = \frac{2.00}{1.44 \times 4} = .35 \text{ sq. in.}$$ Use: $\frac{5}{8}''$ ϕ Bars — 11″ o.c. (from E_2)

$$A_{sBneg.} = \frac{2.40}{1.44 \times 4} = .42 \text{ sq. in.}$$ Use: $\frac{5}{8}''$ ϕ Bars — $8\frac{1}{2}''$ o.c. (from E_1)

The problem could be further extended to slabs under concentrated loads, etc. However, uniformly distributed loads are commonly used and heavy concentrated loads are seldom supported on thin slabs.

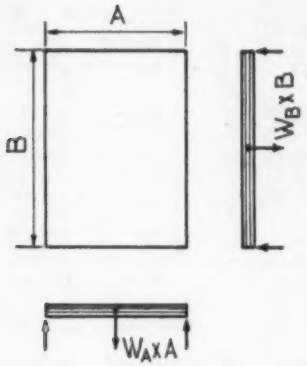
It could be observed from the diagrams that heavy torsional moments occur along the edges of freely supported slabs. While these could be taken care of by special diagonal reinforcings at the corners, such truly simply supported slabs are seldom constructed, the slabs usually resting on monolytically built beams.

The reactions of the slabs act as loadings on the supporting beams and girders, and with sufficient safety it could be stated that the partial loads of w_A and w_B can be used and the reactions of the strips applied as uniformly distributed loads on the beams.

It also should be emphasized that the bending moments along and parallel to the edges are diminishingly small and justify a great reduction of the reinforcing. Thus the main reinforcing should merely be used in the

TABLE 1a—TWO-WAY REINFORCED CONCRETE SLABS

$R = \frac{B}{A}$; $M_{Amax.} = \frac{w A^2}{n_A}$; $M_{Bmax.} = \frac{w B^2}{n_B}$; $w_A = z_A w$; $w_B = (1 - z_A) w$;

Case a	R	$n1_A$	$n1_B$	$z1_A$
 A and B are interchangeable	0.50	169.17	10.57	0.0588
	0.55	125.10	11.35	0.0838
	0.60	94.94	12.30	0.1147
	0.65	75.31	13.44	0.1515
	0.70	61.60	14.79	0.1936
	0.75	51.69	16.35	0.2404
	0.80	43.97	18.01	0.2906
	0.85	38.29	20.15	0.3430
	0.90	34.26	22.36	0.3962
	0 95	30.44	24.79	0.4489
	1.00	27.43	27.43	0.5000
	1.10	22.79	33.37	0.5942
	1.20	19.45	40.34	0.6747
	1.30	17.02	48.60	0.7407
	1.40	15.22	58.45	0.7935
	1.50	13.87	70.22	0.8351
	1.60	12.88	84.43	0.8676
	1.70	12.06	100.77	0.8931
	1.80	11.45	121.69	0.9130
	1.90	10.97	143.00	0.9287

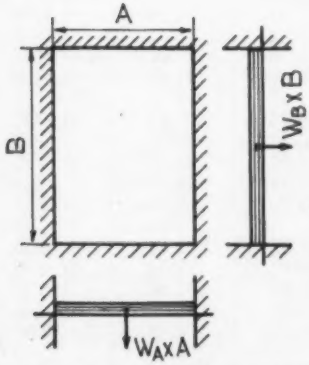
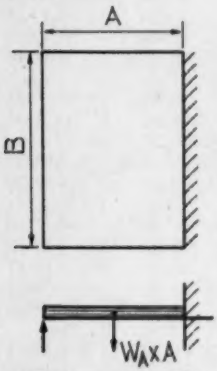
Case b	R	$n2_A$	$n2_B$	$z2_A$
 A and B are interchangeable	0.50	436.53	27.28	0.0588
	0.55	310.15	28.38	0.0838
	0.60	229.50	29.74	0.1147
	0.65	175.97	31.41	0.1515
	0.70	139.24	33.43	0.1936
	0.75	113.30	35.85	0.2404
	0.80	94.51	38.71	0.2906
	0.85	80.60	42.08	0.3430
	0.90	70.10	46.00	0.3962
	0.95	62.04	50.53	0.4489
	1.00	55.74	55.74	0.5000
	1.10	46.77	68.48	0.5942
	1.20	40.90	84.80	0.6747
	1.30	36.89	105.38	0.7407
	1.40	34.08	130.92	0.7935
	1.50	32.04	162.22	0.8351
	1.60	30.54	200.13	0.8676
	1.70	29.40	245.53	0.8931
	1.80	28.52	299.38	0.9130
	1.90	27.75	362.69	0.9287

TABLE 1b—TWO-WAY REINFORCED CONCRETE SLABS

$$R = \frac{B}{A}; \quad M_{A\,max.} = \frac{w\,A^2}{n_A}; \quad M_{B\,max.} = \frac{w\,B^2}{n_B}; \quad w_A = z_A\,w; \quad w_B = (1 - z_A)\,w;$$

Case c	R	$n3_A$	$n3_B$	$z3_A$
 B always parallel to the fixed side	0.50	140.91	11.28	0.1351
	0.55	107.37	12.38	0.1862
	0.60	85.30	13.70	0.2447
	0.65	70.59	15.29	0.3086
	0.70	59.24	17.19	0.3751
	0.75	50.86	19.41	0.4417
	0.80	44.56	21.99	0.5059
	0.85	39.70	24.96	0.5661
	0.90	35.74	28.37	0.6212
	0.95	32.54	32.30	0.6706
	1.00	29.93	36.75	0.7143
	1.10	26.02	47.58	0.7854
	1.20	23.33	61.38	0.8383
	1.30	21.43	78.75	0.8772
	1.40	20.04	100.28	0.9057
	1.50	19.02	126.64	0.9268
	1.60	18.30	158.52	0.9425
	1.70	17.63	196.69	0.9543
	1.80	17.05	239.81	0.9633
	1.90	16.67	295.08	0.9702

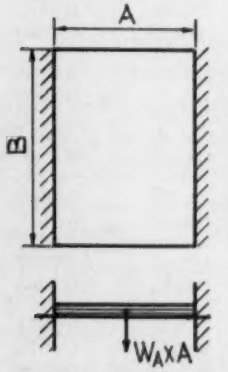
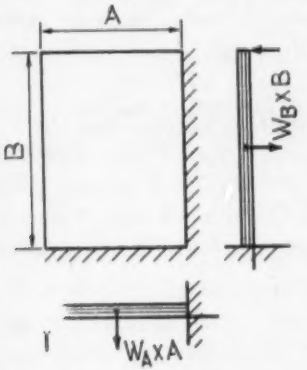
Case d	R	$n4_A$	$n4_B$	$z4_A$
 B always parallel to the fixed sides.	0.50	136.06	12.48	0.2381
	0.55	107.42	14.10	0.3139
	0.60	87.62	16.12	0.3932
	0.65	73.76	18.60	0.4716
	0.70	63.69	21.61	0.5456
	0.75	56.16	25.24	0.6127
	0.80	50.42	29.56	0.6709
	0.85	45.97	34.66	0.7230
	0.90	42.48	40.65	0.7664
	0.95	39.70	47.64	0.8029
	1.00	37.47	55.74	0.8333
	1.10	34.18	75.33	0.8798
	1.20	31.93	101.68	0.9120
	1.30	30.34	134.65	0.9346
	1.40	29.18	175.88	0.9505
	1.50	28.31	226.65	0.9620
	1.60	27.64	288.36	0.9704
	1.70	27.12	362.50	0.9766
	1.80	26.71	450.72	0.9813
	1.90	26.37	554.49	0.9849

TABLE 1c—TWO-WAY REINFORCED CONCRETE SLABS

$$R = \frac{B}{A}; \quad M_{Amax.} = \frac{w A^2}{n_A}; \quad M_{Bmax.} = \frac{w B^2}{n_B}; \quad w_A = z_A w; \quad w_B = (1 - z_A) w;$$

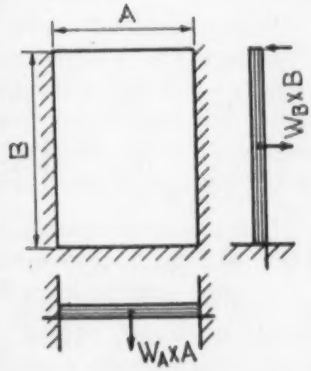
Case e



A and B are interchangeable

R	$n5_A$	$n5_B$	$z5_A$
0.50	271.75	16.98	0.0588
0.55	194.98	17.84	0.0838
0.60	145.73	18.89	0.1147
0.65	112.92	20.16	0.1515
0.70	90.16	21.65	0.1936
0.75	73.99	23.41	0.2404
0.80	62.18	25.47	0.2906
0.85	53.34	27.84	0.3430
0.90	46.58	30.56	0.3962
0.95	41.32	33.65	0.4489
1.00	37.15	37.15	0.5000
1.10	31.09	45.52	0.5942
1.20	27.01	56.01	0.6747
1.30	24.17	69.02	0.7407
1.40	22.12	84.99	0.7935
1.50	20.62	104.38	0.8351
1.60	19.49	127.72	0.8676
1.70	18.62	155.54	0.8931
1.80	17.95	188.41	0.9130
1.90	17.41	226.93	0.9287

Case f



A always parallel to free side

R	$n6_A$	$n6_B$	$z6_A$
0.50	246.42	17.86	0.1111
0.55	186.97	19.12	0.1547
0.60	138.61	20.68	0.2058
0.65	110.30	22.60	0.2631
0.70	90.65	24.92	0.3244
0.75	76.58	27.69	0.3876
0.80	66.24	30.98	0.4503
0.85	58.46	34.84	0.5108
0.90	52.51	39.35	0.5675
0.95	47.86	44.56	0.6196
1.00	44.18	50.57	0.6667
1.10	38.84	65.30	0.7454
1.20	35.27	84.35	0.8057
1.30	32.79	108.24	0.8510
1.40	31.01	138.11	0.8848
1.50	29.71	174.79	0.9101
1.60	28.73	219.29	0.9291
1.70	27.97	272.66	0.9435
1.80	27.38	336.02	0.9545
1.90	26.92	410.58	0.9631

middle half of the slab in both directions, while in the side—strips, not more than half of the theoretically required bars shall be used.

The above developed design formulas are generally used in many European countries with outstanding success, security and economy. This type of floor construction is specially suitable for apartment buildings and light manufacturing plants, where the reduced formwork and the elimination of a suspended ceiling brings about a great saving. There is nothing unusual in using panels up to twenty feet square, hiding the beams in the partition walls in case of apartment buildings, while in case of industrial buildings the absence of concentrated loads on girders greatly reduces the necessary depth and reinforcings.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1945 for publication in the JOURNAL for June 1945.

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Discussion of a paper by Paul Rogers:

Two-Way Reinforced Concrete Slabs*

By J. B. MACPHAIL† and AUTHOR

The author has performed a useful service in making the work of Dr. Marcus available in English, but his presentation of it has not the clarity which the subject deserves. Some of the definitions appear, on comparison with the numerical examples, to need amendment; and the notation is unnecessarily complicated.

The writer therefore proposes to suggest some changes which may help others in getting a better understanding of a useful paper. If any seem trivial, it should be remembered that many readers, particularly the younger ones, begin study of a paper with the presumption that the author is right, and they may go through some anxious moments before they feel sufficiently sure of their ground to correct him.

In the definitions, M_A and M_B seem to mean maximum positive moments in the A and B directions per unit width of uncut slab, using the customary convention that positive moments in a slab make it concave on top. Likewise m_A and m_B seem to be maximum positive moments on strips of unit width cut at the middle of the slab and of lengths A and B, as shown in Fig. 1, when subjected to loads w_A and w_B respectively and the end restraints appropriate to the case considered. The suffix or subscript *max* can then be omitted, because the paper considers no positive moments other than maximum ones.

The coefficients ϕ and γ appear to apply only to maximum positive moments; and not to negative moments, nor to positive moments other than maximum ones.

Briefly, the moments considered are particular ones, under restrictions of place, magnitude and sign, but the definitions do not so describe them.

*ACI Journal, Sept. 1944; *Proceedings* V. 41, p. 21.
†The Shawinigan Engineering Co., Ltd., Montreal, Que.

In the tables 1a, 1b and 1c, the use of italic alphabetical subscripts with n and z instead of numerical ones would bring the notation into conformity with the alphabetical designations of the cases and with the subscripts in the test, and would save the reader from searching the text in vain for a definition of the numerical subscripts.

To drop the subscripts for cases entirely would be still better, both in tables and test. There are plenty of precedents to support such a proposal.

Torsional moments seem not to reduce negative moments, so moments at edges which are fixed appear to be calculated on the basis of strips of unit width as $-w_A A^2/8$ or $-w_A A^2/12$, according as the opposite edge is free or fixed: with corresponding expressions for the other direction.

There are some other minor discrepancies but they will not cause difficulty.

The writer unfortunately has not convenient access to a copy of Dr. Marcus' book, so he will be glad to have either confirmation or correction of this statement of his understanding of the matter.

AUTHOR'S CLOSURE

Mr. Macphail points out that some of the notations of the paper are not clear and are misleading.

M_A or M_B could be substituted instead of M_A *max* and M_B *max*, although it is the author's intention to emphasize that these Moments are the maximum. The purpose of the paper is to provide for a working formula for practical designing and not a mathematical analysis. Thus the solutions are made only for maximum moments. The moment-diagram follows the usual parabolic curve and it is an easy matter to establish moments at different points.

The negative moments act at the supporting beams, where the torsional moments have zero value. (See diagrams) Obviously these negative moments cannot be reduced, and their value is obtained by standard procedure.

The author wishes to mention that he prepared the paper with the approval and suggestions of Dr. Henri Marcus himself, whose presence in this country is a great benefit to engineering science.

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Reactivity of Aggregate Constituents in Alkaline Solutions*

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SYNOPSIS

The relative order of reactivity of 12 types of aggregate constituents to alkali hydroxide solutions was determined, by means of an accelerated test at 122 C, to be as follows: opal, chalcedony, pitchstone, rhyolite, basalt, magnesite, obsidian, calcite, limestone, dolomite, microcline, and oligoclase. The carbonates were more reactive to KOH than NaOH. The feldspars and glass bearing rocks were more susceptible to attack by NaOH. Reactivity to carbonate and sulfate solutions was found to be negligible under the conditions chosen. Quantitative data on the solubility of the materials in hydroxide and carbonate solutions at 21 C are given. These agree with the results of the accelerated test. These data shed light on alkali reactivity under the conditons of the laboratory tests used, but make no attempt to predict expansion of concrete on the basis of these tests alone.

1. INTRODUCTION

The discovery of concrete failures when high-alkali cements have been used in conjunction with certain types of aggregates has led to considerable investigation into the cause and mechanism, as well as methods of prevention, of these failures [1,2]*. Recent investigators believe that these failures have been due to reaction between the alkalies (Na_2O and K_2O) originally present in cement and certain constituents of the aggregate. The exact nature of the reactions is not understood, although deposits of alkali silicate gel generally accompany this type of failure.

As a part of a general study at the National Bureau of Standards of the cause and mechanism of the alkali-aggregate reaction, various reactivity tests were made. The methods used provide data on the decomposability of certain aggregate materials in the presence of alkaline solutions and on the composition of the material dissolved by the alkaline solutions.

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†National Bureau of Standards.
*See references at the end of this paper.

No claim is made that these tests by themselves should be used as a criterion to determine fitness of aggregates for use in concrete.

2. DESCRIPTION OF MATERIALS

In selecting aggregate constituents for these studies rocks and minerals were included, 1) which had been named in the literature as being reactive with alkalies, 2) which were believed to be unreactive, and 3) whose alkali reactivity was unknown. These were basalt (Mount St. Helens, Wash.), calcite (Taos Co., N. Mex.), chalcedony (Serra do Mar, Brazil), dolomite (Tuckahoe, N. Y.), limestone (western Pennsylvania), magnesite (Muntodi, Greece), microcline feldspar (Bathurst, Ont.), obsidian (Lake Co., Oregon), oligoclase feldspar (Mitchell Co., N. C.), opal (Virgin Valley, Nev.), pitchstone (Magdalena Mts., N. Mex.) and rhyolite (White Pine Co., Nev.). A few tests were also made on chert (Potomac River gravel, Maryland-Virginia), opaline chert (San Luis Obispo, Calif.), phyllite (Buck Dam, Va.), quartz (Bedford, N. Y.), and siliceous magnesian limestone (Paso Robles, Calif.) A brief mineralogical description is given in Table 1 of the paper by Parsons and Insley[3] of the calcite, chalcedony, dolomite, microcline feldspar, obsidian, oligoclase feldspar, opal, pitchstone, rhyolite, opaline chert, phyllite, and siliceous magnesian limestone. The following description of the remaining materials has been supplied by Willard H. Parsons:

Basalt—Vesicular structure; about 75 per cent labradorite feldspar; rest augite, partly altered olivine, and magnetite.

Limestone—Fine-grained calcite limestone; about 5 per cent kaolin, 1 per cent quartz, trace of pyrite.

Magnesite—Very fine-grained, dense, crystalline magnesite; 3 to 5 per cent kaolin, trace of limonite.

Chert—Brown and white cherts; mixtures of microcrystalline quartz and chalcedony in varying proportions; traces of calcite and limonite.

Quartz—Coarsely crystalline, pale-smoky quartz; no visible impurities.

3. TEST METHODS AND RESULTS

1. Change in weight after exposure to various solutions

(*a*) *Alkali hydroxide solutions at 21C*—Pieces of 12 of the minerals and rocks weighing 3 to 5 grams each were soaked in water a few hours, dried with a towel, and immediately weighed. Each sample was placed in a test tube and covered with 15 ml of 5 per cent NaOH, 5 per cent KOH, or saturated $Ca(OH)_2$ solution. The test tubes were then stoppered, sealed with ceresin, and placed in a constant-temperature room at 21 C (70 F) for 1 year. At the end of 1 year the tubes were opened and the specimens removed, rinsed well with water, dried with a towel, and weighed. The percentage change in weight is shown in Table 1.

In both NaOH and KOH opal was the most reactive, chalcedony exhibited the second highest reactivity, and magnesite and pitchstone followed chalcedony in order of weight lost. All gained in weight in $Ca(OH)_2$ solution.

TABLE 1 — PERCENT CHANGE IN WEIGHT OF AGGREGATE CONSTITUENTS AFTER STANDING 1 YEAR IN CONTACT WITH SOLUTION AT 21C (70F)

	% loss with 5% NaOH	% loss with 5% KOH	% gain with sat. $Ca(OH)_2$
Basalt	.0	.0	1.5
Calcite	.0	.0	.1
Chalcedony	8.0	6.2	.3
Dolomite	.0	.0	.4
Limestone	.0	.0	1.2
Magnesite	3.1	2.7	.5
Microcline	.0	.0	.3
Obsidian	.5	.0	.2
Oligoclase	.0	.0	.3
Opal	54.0	13.8	1.6
Pitchstone	3.6	.3	.5
Rhyolite	1.0	.0	1.5

(*b*) *Alkali hydroxide solutions at 122C*—Accelerated tests of susceptibility of aggregate materials to alkaline attack being desirable, the experiments described below were conducted. An ordinary pressure cooker of the common kitchen variety, equipped with a pressure gauge, was used as an autoclave. Water to a depth of about one inch was poured into the cooker. Following the procedure described in the preceeding section, the samples were prepared and placed in test tubes containing the alkali solutions. Loose cotton plugs were put in the mouths of the tubes. The test tubes were supported in 600 ml beakers and each beaker placed on a ceramic ring in the bottom of the pressure cooker. Heat was supplied by means of a Meker burner.

Steam was allowed to escape for several minutes before the valve was closed, to insure exhaustion of air and carbon dioxide. After closing the valve the pressure soon rose to 15 pounds above atmospheric pressure, which was taken as the starting point. This was equivalent to a temperature of 122 C. By adjustment of the burner the pressure was maintained within 1 pound of 15 pounds for the required period. The burner was then turned off, the cooker cooled for 15 minutes, the steam allowed to escape slowly, and the samples removed. The solutions were then poured off and the samples washed, dried, and weighed as described in the preceding section. In the case of the very reactive materials, any gel-like substance was gently washed away, care being taken not to break off any pieces of the aggregate itself.

The aggregate materials were treated with NaOH and KOH solutions of various strengths by weight from 1 to 50 per cent. Two and three hours were the periods of heating in contact with 1 percent solutions, in all other cases the treatment was for 1, 2 and 4 hours. In each case, except those marked with an asterisk in Table 2, the same specimen was

TABLE 2—PERCENT LOSSES OF ORIGINAL WEIGHT OF AGGREGATE CONSTITUENTS IN NaOH AND KOH SOLUTIONS AT 122C (252F).

Test No.	Solution; % NaOH or KOH	Time in Hours	Per cent loss of original weight of aggregate constituent											
			Basalt	Calcite	Dolomite	Limestone	Magnesite	Microcline	Obsidian	Oligoclase	Opal	Pitchstone	Chalcedony	Rhyolite
							NaOH							
1	1	2	0.0	0.0	0.0	0.0	0.3	0.0	0.6	0.0	4.3	0.7	1.4	0.0
2	1	3	.0	.0	.0	.0	.0	.0	.0	.0	5.9	.2	1.3	.6
3	5	1	1.4	.0	.0	.0	.4	.0	1.6	.0	3.9	.7	10.7	1.1
4	5	2	.4	.0	.0	.0	.1	.0	.3	.0	*26.2	1.4	* 9.4	.8
5	5	4	.0	.0	.0	.0	.4	.0	.4	.0	35.7	1.8	7.2	1.9
6	10	1	.6	.0	.0	.1	.2	.0	.3	.1	*32.8	1.4	*46.3	1.6
7	10	2	.0	.0	.1	.0	.1	.1	.2	.0	44.3	1.4	13.4	.9
8	10	4	1.3	.0	.0	.0	.6	.0	.9	.0	*73.8	1.6	19.7	1.9
9	15	1	.0	.1	.0	.0	.3	.0	.8	.1	*47.5	1.6	*21.4	1.6
10	15	2	.7	.3	.2	.0	.6	.0	1.0	.0	47.7	3.0	61.5	1.3
11	15	4	.3	.0	.0	.0	.6	.1	.5	.0	*61.1	3.2	*30.0	2.4
12	20	1	.0	.0	.0	.0	.4	.0	2.1	.0	29.5	1.8	*29.7	1.2
13	20	2	1.7	.4	.1	.0	1.4	.0	.7	.0	*62.3	7.7	47.0	5.5
14	20	4	.0	.0	.0	.0	.0	.0	.9	.0	*53.5	.0	*36.6	.0
15	25	1	.9	.4	.3	.3	.3	.0	.6	.0	25.0	1.9	12.7	1.6
16	25	2	1.6	.4	.5	.6	1.2	.2	.7	.2	*43.5	6.0	18.8	6.5
17	25	4	1.0	.2	.0	.0	.8	.2	1.1	.0	40.5	4.3	16.4	3.1
18	30	1	.0	.5	.4	.4	.0	.1	.7	.0	*70.0	.0	*18.6	.4
19	30	2	2.2	.9	1.0	1.0	.6	.2	.6	.3	29.4	7.9	27.1	7.2
20	30	4	.6	.4	.4	.2	.8	.0	1.0	.0	*78.6	3.4	28.7	5.6
21	35	1	.5	.7	.6	1.2	.0	.4	.4	.2	*43.5	not det.	7.1	4.0
22	35	2	.8	.2	.0	.4	.0	.3	.5	.2	47.9	*5.4	9.5	3.3
23	35	4	2.4	.4	.3	.6	.0	.3	.9	.0	*83.6	7.8	*54.9	11.2
24	40	1	2.5	.4	.2	.4	.4	.2	.3	.2	*54.7	4.1	14.3	* .5
25	40	2	3.2	.0	.4	.4	1.7	.4	.6	.4	38.6	7.4	*23.3	10.2
26	40	4	.7	.4	.0	.0	.0	.5	1.0	.4	not det.	1.8	43.9	1.9
27	45	1	3.0	.3	.2	.4	.9	.3	.5	.3	*78.9	13.0	*21.0	2.9
28	45	2	2.1	.0	.0	.0	.0	.5	.5	.3	*80.3	8.6	19.6	3.3
29	45	4	2.0	.0	.0	.2	.0	.9	1.2	.3	not det.	*13.4	12.7	4.1
30	50	1	2.0	.0	.4	.3	.0	.3	.4	.4	*35.6	1.3	*6.8	1.7
31	50	2	2.2	1.8	1.4	.6	1.4	.7	.7	.3	*67.1	.0	9.3	2.5
32	50	4	2.3	.8	1.1	.3	1.3	.8	1.0	.6	27.9	9.5	9.6	19.8

			KOH											
33	1	2	.0	.0	.0	.0	.9	.0	.0	.0	3.2	.3	.3	.0
34	1	3	.0	.0	.0	.0	.0	.0	.0	.0	3.1	.1	.3	.0
35	5	1	.4	.0	.0	.0	.4	.0	.0	.0	13.1	.2	1.8	.3
36	5	2	.0	.0	.0	.0	.9	.0	.0	.0	18.2	.2	4.1	.5
37	5	4	.8	.0	.0	.0	.5	.0	.0	.0	25.6	.4	9.4	.5
38	10	1	.0	.0	.0	.0	.4	.0	.1	.1	*60.1	.4	9.3	.8
39	10	2	.1	.0	.0	.0	.1	.0	.1	.0	18.8	.6	14.4	.9
40	10	4	.3	.0	.0	.0	1.0	.0	.1	.0	*46.9	.8	18.0	1.1
41	15	1	.4	.0	.1	.0	.4	.1	.1	.0	*37.2	1.0	6.9	1.7
42	15	2	.0	.0	.0	.0	.7	.0	.2	.0	40.7	.0	18.9	1.2
43	15	4	.7	.0	.0	.0	.6	.0	.1	.1	*40.5	1.5	21.1	1.2
44	20	1	.0	.3	.4	.3	.2	.2	.0	.0	35.5	1.4	18.6	1.1
45	20	2	.3	.2	.3	.0	1.7	.0	.2	.0	23.8	3.6	30.5	2.9
46	20	4	.0	.1	.1	.0	.0	.0	.2	.0	*90.0	2.4	*26.1	1.1
47	25	1	.0	.4	.3	.5	.0	.1	.0	.0	*36.0	1.8	9.4	2.0
48	25	2	1.2	.1	.2	.0	1.8	.0	.1	.0	31.0	4.9	19.3	3.2
49	25	4	.0	.0	.0	.0	.6	.2	.6	.0	26.0	3.8	23.5	2.6
50	30	1	.0	.6	.4	.7	.0	.0	.1	.0	*75.0	.0	*20.7	.4
51	30	2	1.8	1.0	.9	.7	1.6	.2	.3	.0	*91.8	6.0	33.7	4.0
52	30	4	.4	.3	.3	.3	.4	.0	.3	.0	*64.4	2.5	30.2	3.1
53	35	1	.2	1.1	1.1	1.2	.4	.0	.1	.0	21.2	.0	*11.8	2.7
54	35	2	.0	1.0	.7	1.0	.0	.0	.2	.0	*77.7	1.2	63.5	1.1
55	35	4	1.5	.6	.6	.7	.3	.0	.2	.0	*90.3	3.6	*58.7	4.2
56	40	1	.0	.6	.5	.6	.4	.0	.2	.0	*60.5	3.5	* 5.0	.0
57	40	2	1.4	.6	.5	.5	1.7	.2	.1	.0	*73.4	5.9	39.9	5.2
58	40	4	.0	.7	.4	.5	.0	.0	.7	.0	*91.4	.0	44.2	.6
59	45	1	.5	.4	.2	.4	.4	.1	.1	.0	*72.4	2.3	*4.8	3.6
60	45	2	2.2	.6	.8	.6	2.1	.2	.2	.0	*87.9	6.2	36.8	6.3
61	45	4	1.5	.4	.4	.6	.4	.0	.4	.0	*92.7	2.8	30.4	2.3
62	50	1	.0	.2	.0	.2	.0	.0	.0	.0	*52.0	.2	10.3	.0
63	50	2	.0	.4	.4	.5	.0	.0	.3	.2	*86.8	*2.8	*16.3	2.4
64	50	4	.6	.3	.4	.5	.4	.0	.3	.2	*56.1	10.3	54.1	2.0

*A new sample was used for these determinations. In all other cases, the sample used in the preceding test was used again.

used in succeeding tests. Thus after treatment for 2 hours with the 1 per cent NaOH solution and weighing, what remained of the specimen was subjected to the same solution for 2 hours, then successively treated for 1, 2, and 4 hours with a 5 per cent NaOH solution. The 1, 2, 4 hour cycle was then repeated at higher NaOH concentrations. The same procedure was used with KOH solutions.

The percentage losses in weight were calculated on the basis of the original weights of the samples tested and are recorded in Table 2. When new samples were introducted because of disintegration or accidental loss, the values are marked with an asterisk, and subsequent calculations were based on the weight of the new specimen.

Opal and chalcedony are by far the most reactive of any of the materials. Pitchstone and rhyolite, though many times less reactive than opal, are definitely more reactive than the remaining materials. In an effort to obtain an average order of reactivity, the percentages of weight lost by each sample of rock and mineral throughout the series of tests covered by Table 2 were added. These totals were arranged in descending order as shown in Table 3, giving an approximate order of reactivity. The more reactive specimens, which had to be replaced several times, thus show totals of over 100 per cent loss.

By observation of Table 3, it may be seen that basalt, microcline, obsidian, oligoclase, pitchstone, and rhyolite lost more weight with NaOH than with KOH. Calcite, dolomite, limestone, and magnesite lost more on treatment with KOH. Opal and chalcedony were about as reactive with one as with the other, although there was a slight tendency for opal to be more reactive with KOH.

To see how the accelerated method of test would agree with the etch test used by Parsons and Insley[3], five samples each of chert, opaline chert, phyllite, quartz, and siliceous magnesian limestone, were subjected to 10 per cent NaOH for 1 hour at 122 C and then for another 2-hour period. The siliceous magnesian limestone was given an additional 4-hour treatment in 20 per cent NaOH solution. Samples of obsidian and magnesite were tested at the same time for comparison. The changes in weight are reported in Table 4. In comparing these results with those in Table 2, it should be borne in mind that the specimens treated with 10 per cent NaOH solution as shown in Table 2, had previously been subjected to the action of 1 and 5 per cent NaOH solutions.

The order of reactivity (omitting siliceous magnesian limestone) is: opaline chert, Potomac chert, magnesite, obsidian, phyllite, quartz.

The siliceous magnesian limestone gained slightly in weight, even in 20 per cent NaOH for 4 hours. In an effort to estimate the amount of "soluble silica" or opal present in the siliceous magnesian limestone, samples of the rock were powdered to pass a No. 200 sieve and digested with 5

TABLE 3 — APPROXIMATE ORDER OF REACTIVITY OF AGGREGATE CONSTITUENTS IN ALKALI HYDROXIDE SOLUTIONS

Order of reactivity with 1 to 50% NaOH solutions	*Total percent loss of weight	Order of reactivity with 1 to 50% KOH solutions	*Total percent loss of weight
1. Opal	1373.6	1. Opal	1643.3
2. Chalcedony	689.9	2. Chalcedony	692.3
3. Pitchstone	122.3	3. Pitchstone	70.7
4. Rhyolite	110.6	4. Rhyolite	59.0
5. Basalt	36.4	5. Magnesite	18.3
6. Obsidian	23.0	6. Basalt	14.3
7. Magnesite	14.8	7. Calcite	9.9
8. Calcite	8.6	8. Limestone	9.8
9. Dolomite	7.6	9. Dolomite	9.0
10. Limestone	7.4	10. Obsidian	5.3
11. Microcline	6.5	11. Microcline	1.3
12. Oligoclase	4.3	12. Oligoclase	.6

Combined order of reactivity with NaOH and K OH solutions

(Sums of the values above)	Total per cent loss of weight
1. Opal	3016.9
2. Chalcedony	1382.2
3. Pitchstone	193.0
4. Rhyolite	169.6
5. Basalt	50.7
6. Magnesite	33.1
7. Obsidian	28.3
8. Calcite	18.5
9. Limestone	17.2
10. Dolomite	16.6
11. Microcline	7.8
12. Oligoclase	4.9

*Since it was necessary to replace the more reactive rocks and minerals several times, some show losses well over 100 per cent of the weight of the original specimen.

per cent Na_2CO_3 solution for 15 minutes on the steam bath (4). An average of 12.3 per cent "soluble silica" and 11.7 per cent "insoluble silica" was found*. These values agree approximately with the estimated opal and chalcedony content of the rock[3].

The results shown in Tables 1-4 should not be considered quantitative, because of differences in the amount of active surface exposed on specimens of the same and different aggregates. Surface effects would be even more important after some reaction had taken place. For some materials such as opal, a surface coating of gel-like reaction products retarded further reaction, whereas in other materials lack of homogeneity of the sample was a factor. Weight loss, therefore, should not be depended upon solely, in evaluating reactivity of aggregates toward alkalies.

*Analysis made by Cyril Crocker of the Cement Laboratory of the National Bureau of Standards.

TABLE 4—PERCENT CHANGE IN WEIGHT OF SAMPLES OF AGGREGATE MATERIALS AFTER TREATMENT WITH NaOH SOLUTION AT 122C (252F)

Percent NaOH solution	Time in hours	Aggregate constituent	Percent change* in weight of samples					
			Sample Numbers					
			1	2	3	4	5	Avg.
10	1	Chert (Potomac River)	2.5	2.3	1.5	.8	.5	1.5
10	2	Chert (Potomac River)	16.0	16.4	8.4	6.0	2.7	9.9
10	1	Opaline chert	28.4	41.5	16.5	76.7	54.6	43.5
10	2	Opaline chert	33.2	49.4	34.2	23.3	34.1	34.8
10	1	Phyllite	.0	.0	.0	.2	.0	.0
10	2	Phyllite	.3	.4	.2	.5	.5	.4
10	1	Quartz	.0	.0	.0	.0	.0	.0
10	2	Quartz	.0	.0	.0	.0	.0	.0
10	1	Si-Mg limestone	+.3	.0	+.1	+.3	+.2	+.2
10	2	Si-Mg limestone	+.2	+.1	+.1	+.2	+.1	+.1
20	4	Si-Mg limestone	+.8	+.7	+.8	+.8	+.3	+.7
10	1	Obsidian	.3	.2	.2	.2	.4	.3
10	2	Obsidian	.2	.6	.8	.5	.9	.6
10	1	Magnesite	2.5	2.0	1.6	1.3	2.0	1.9
10	2	Magnesite	1.2	.8	.9	1.1	1.1	1.0

*The values represent losses in weight except those for siliceous magnesian limestone which represent gains in weight.

TABLE 5—PERCENT LOSS IN WEIGHT OF SAMPLES AFTER TREATMENT AT 122C (252F) WITH MIXTURES OF ALKALI HYDROXIDES AND CARBONATES FOR 4 HOURS.

	Percent loss in weight of samples in solutions containing:	
	10% NaOH + 10% Na_2CO_3	10% K OH + 10% K_2CO_3
Basalt	.3	.6
Calcite	.0	.0
Dolomite	.0	.0
Limestone	.0	.0
Magnesite	.0	.7
Microcline	.0	.0
Obsidian	.7	.0
Oligoclase	.0	.0
Opal	70.0	71.8
Pitchstone	.0	1.4
Chalcedony	30.9	25.5
Rhyolite	.0	2.2

(*c*) *Alkali carbonate solutions at 122C*—Using exactly the same technique as previously mentioned, the reactivity of the first 12 aggregate constituents with 25 per cent sodium and with 25 per cent potassium carbonate solutions was tested. At 122 C for 4 hours, only opal showed any loss in weight (6.3 per cent in Na_2CO_3 and 7.7 per cent in K_2CO_3). New samples of the materials were treated with solutions containing 10 per cent NaOH plus 10 per cent Na_2CO_3 and with 10 per cent KOH plus 10 per cent

K_2CO_3. The per cent losses are shown in Table 5. From these experiments, the evidence seems to indicate that alkali carbonates are not very reactive with the specimens studied. The weight losses shown in Table 5 are of the order of magnitude of those obtained with the alkali hydroxides alone.

(*d*) *Alkali sulfate solutions at 122C*—The same tests using 10 per cent Na_2SO_4 and also using 10 per cent K_2SO_4 were made with the same materials. Losses were insignificant.

2. Composition of material dissolved by various solutions

Several series of experiments were conducted to determine the materials dissolved from different rocks and minerals by solutions of NaOH, KOH, and each plus its respective carbonate. Conditons were chosen that would preclude solution of SiO_2 or other oxides from the containers. Carbon dioxide was excluded by sealing the containers with ceresin. The temperature was held at 21 C by conducting the experiments in a constant-temperature room

(*a*) *NaOH solution*—In the first series, 75 gram samples of each of 6 aggregate materials were ground until all the material passed a No. 325 sieve. Two grams of each of these were placed in 30 ml test tubes made from Lucite and the samples covered with 25 ml portions of 15 per cent NaOH solution. The tubes were closed by rubber stoppers, sealed with ceresin, and their contents agitated by constant rotation for 24 hours at 21 C. On opening the tubes the residue was generally observed to be caked in the bottom. The solution was diluted, filtered into concentrated HCl, and the filtrate analyzed for SiO_2, R_2O_3, CaO, and MgO. The residue was washed back into the test tube and the shaking was repeated for 7 more days, then filtered again and the filtrate analyzed. This was followed by a 28 and then a 35-day treatment. The analyses of the various filtrates are given in Table 6. These values are expressed as percentages of the original weight of the samples. Fig. 1 shows the cumulative values for SiO_2 in solution at the end of each period.

If the percent of the original sample removed as SiO_2 is used as the criterion, the order of reactivity of these 6 materials toward 15 per cent NaOH then is: pitchstone, obsidian, rhyolite, basalt, microcline, and oligoclase. This, it will be noted, is the same order for these 6 materials as obtained in the experiments summarized in Table 3 with one exception, that the positions of rhyolite and obsidian have been reversed. There was considerable extraction in the final 35-day treatment, showing that there was little tendency during the earlier periods for completion of the extraction of any of the constituents.

(*b*) *KOH solution*—Two gram samples each of basalt, obsidian, pitchstone, rhyolite, and microcline, ground to pass a No. 325 sieve, were

TABLE 6—PERCENTAGES OF ORIGINAL SAMPLE FOUND IN SOLUTION AS OXIDES AFTER TREATMENT WITH A 15 PERCENT NaOH SOLUTION AT 21C (70F).

Sample	Time in days*	Percent of original sample found in extract as			
		SiO_2	R_2O_3	CaO	MgO
Basalt	1	.3	.2	.0	.0
Basalt	7	.7	.2	.0	.0
Basalt	28	1.4	.2	.0	.2
Basalt	35	1.0	.3	.1	.1
Obsidian	1	.9	.3	.0	.0
Obsidian	7	3.5	.9	.1	.1
Obsidian	28	8.4	1.8	.0	.1
Obsdian	35	11.0	2 2	.1	.1
Pitchstone	1	1.0	3	.0	.0
Pitchstone	7	5.6	1 2	.1	.0
Pitchstone	28	10.3	1 8	.0	.1
Pitchstone	35	11.3	2.0	.2	.1
Rhyolite	1	1.0	.4	.0	.0
Rhyolite	7	3.2	.9	.0	.1
Rhyolite	28	6.3	1.3	.0	.2
Rhyolite	35	8.9	1.7	.2	.2
Microcline	1	.2	.1	.1	.0
Microcline	7	.2	.1	.1	.0
Microcline	28	.2	.2	.0	.2
Microcline	35	.3	.2	.1	.3
Oligoclase	1	.1	.2	.0	.0
Oligoclase	7	.2	.2	.0	.0
Oligoclase	28	.1	.2	.0	.1
Oligoclase	35	.2	.1	.2	.2

*Time given is additional to that of previous test or tests.

placed in 250 ml Erlenmeyer flasks which had been coated on the inside with a thick layer of ceresin. One hundred ml of 15 per cent KOH was poured in each, the flasks closed with rubber stoppers and sealed with ceresin. These flasks were then placed on a shaking machine which imparted a swirling action to the contents. This gave fairly good agitation and prevented the caking of the residues such as occurred in the experiments made in Lucite tubes. They were allowed to shake 1 day in a constant-temperature room at 21C. The alkaline solutions were then diluted, filtered into concentrated HCl, and the residues thoroughly washed. The residues were discarded and the filtrates analyzed for SiO_2, R_2O_3, CaO, MgO, and SO_3.

Starting with new 2 gram samples in freshly waxed flasks, the above procedure was repeated for 7 days and again with fresh samples for 47 days. The percentages of the constituents found in the KOH extracts, based on the original 2 gram samples, are shown in Table 7 and the values for SiO_2 are also shown in Fig. 2.

The order of reactivity based on per cent of original sample removed as SiO_2 in 47 days by 15 per cent KOH, then is: pitchstone, obsidian, rhyolite, microcline, and basalt.

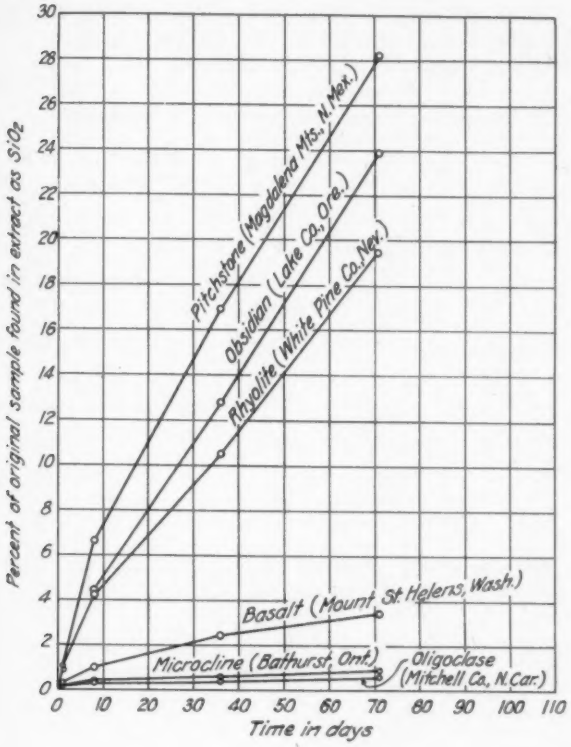
Percent of original sample found in extract as SiO_2
Time in days
Pitchstone (Magdalena Mts., N. Mex.)
Obsidian (Lake Co., Ore.)
Rhyolite (White Pine Co., Nev.)
Basalt (Mount St. Helens, Wash.)
Microcline (Bathurst, Ont.)
Oligoclase (Mitchell Co., N. Car.)

Fig. 1

Fig. 2

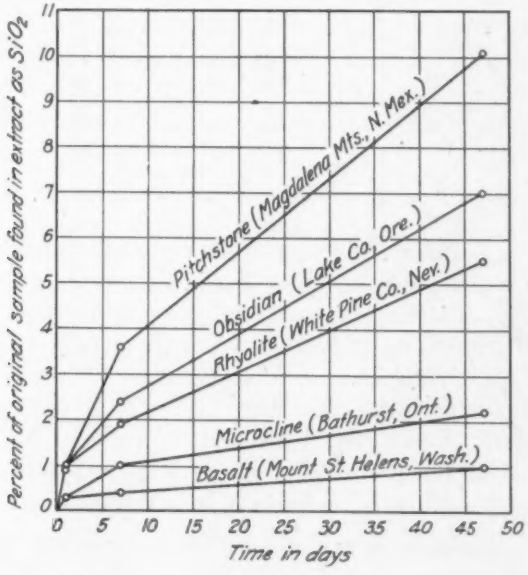
Percent of original sample found in extract as SiO_2
Time in days
Pitchstone (Magdalena Mts., N. Mex.)
Obsidian (Lake Co., Ore.)
Rhyolite (White Pine Co., Nev.)
Microcline (Bathurst, Ont.)
Basalt (Mount St. Helens, Wash.)

TABLE 7—PERCENTAGE OF VARIOUS OXIDES DISSOLVED FROM MATERIALS BY A 15 PERCENT SOLUTION OF K OH AT 21C (70F). NEW SAMPLES USED FOR EACH TEST.

Aggregate Constituents	Time in days	Percent of original sample found in extract as				
		SiO_2	R_2O_3	CaO	MgO	SO_3
Basalt	1	.3	.1	.0	.0	.0
"	7	.4	.1	.0	.1	.0
"	47	1.0	.3	.0	.0	.0
Obsidian	1	1.0	.2	.0	.0	.0
"	7	2.4	.5	.0	.1	.0
"	47	7.0	1.4	.0	.0	.0
Pitchstone	1	.9	.1	.2	.0	.0
"	7	3.6	.6	.0	.1	.0
"	47	10.1	1.9	.0	.0	.1
Rhyolite	1	1.0	.1	.1	.0	.0
"	7	1.9	.4	.0	.1	.0
"	47	5.5	1.3	.0	.0	.2
Microcline	1	.3	.2	.1	.0	.0
"	7	1.0	.7	.1	.0	.0
"	47	2.2	.2	.0	.1	.1

TABLE 8—PERCENTAGES OF VARIOUS OXIDES DISSOLVED AT 21C (70F) BY SOLUTIONS CONTAINING ALKALI HYDROXIDES AND CARBONATES.

Solution: Percent				Material	Time in days	Percent of original sample found in extract as oxides				
NaOH	Na_2CO_3	K OH	K_2CO_3			SiO_2	R_2O_3	CaO	MgO	SO_3
15	5	—	—	Basalt	46	10.9	2.7	.8	.3	.0
15	5	—	—	Obsidian	46	28.4	4.6	.1	.0	.0
15	5	—	—	Pitchstone	46	21.9	4.4	.0	.0	.0
15	5	—	—	Rhyolite	46	18.8	4.3	.4	.1	.0
15	5	—	—	Microcline	46	12.0	1.3	.1	.0	.0
—	—	15	5	Basalt	65	1.6	.6	.2	.0	.0
—	—	15	5	Obsidian	65	10.4	2.1	.1	.0	.0
—	—	15	5	Pitchstone	65	14.0	2.4	.1	.1	.1
—	—	15	5	Rhyolite	65	7.9	1.8	.1	.0	.1
—	—	15	5	Microcline	65	.6	.5	.1	.0	.0

(*c*) *Alkali hydroxide plus carbonate solutions*—Two gram samples of basalt, obsidian, pitchstone, microcline, and rhyolite, ground to pass a No. 325 sieve, were placed in ceresin-lined Erlenmeyer flasks as before. Then 100 ml of a solution containing 15 per cent NaOH plus 5 per cent Na_2CO_3 was poured into each flask. They were stoppered and sealed as previously described and shaken for 46 days at 21 C. Another set treated with 15 per cent KOH plus 5 per cent K_2CO_3 solution was shaken 65 days. The percentages of the original 2 gram samples, found by analysis of the filtrates, are shown in Table 8.

Based on SiO_2 removed, the order of reactivity with 15 per cent NaOH plus 5 per cent Na_2CO_3, is: obsidian, pitchstone, rhyolite, microcline,

basalt. Reactivity with 15 per cent KOH plus 5 per cent K_2CO_3 proceeded in the order: pitchstone, obsidian, rhyolite, basalt, microcline.

4. DISCUSSION

The fact that certain constituents of a rock are reactive with concentrated alkali hydroxide solutions at 122C does not necessarily mean that they would be reactive with the solutions in concrete made with high-alkali cements at lower temperatures. For example, chalcedony is very reactive in the present series of tests, but has not been associated with alkali-aggregate reactivity in the literature. This mineral is an abundant constituent of some cherts which have good service records as concrete aggregates. In alkali etching tests [3], chalcedony, although strongly etched at a higher temperature (90C) was only very slightly reactive at lower temperatures (20 to 40C); chalcedony appears, therefore, to have a behavior which changes markedly with increased temperature.

The results obtained in this study, in most cases, agree with the microscopical methods of Parsons and Insley [3]. All of the aggregates which they found etched at higher temperatures lost considerable weight in the experiments here described, with the notable exception of siliceous magnesian limestone. Numerous cases of expansive failure in concrete have been traced to the presence, in certain aggregates, of this siliceous magnesian limestone [5]. The reactivity of the limestone, however, is not satisfactorily indicated by the autoclave method, since the specimens tested gained in weight. It may be that the texture or other characteristics of this type of rock are such that the alkali silicate which forms on the interior cannot diffuse through the rock and escape. This factor might cause a gain in weight that would more than compensate for the loss from the surface.

Some samples of Potomac River chert, which were unetched in the tests of Parsons and Insley [3], lost considerable weight in the present tests. This appears due to the presence of chalcedony in some samples*. The reactivity to alkali was in the same order as the amount of chalcedony present. The samples of Potomac River chert used by Parsons and Insley in their test, contained only a small amount of chalcedony†.

Although the concrete of Buck Dam in which phyllite was used as the aggregate has shown a poor service record [6], phyllite exhibited little reactivity in this study. It was also found unreactive in the alkali etch tests [3].

*A microscopical examination of the 5 test samples was made by W. H. Parsons, National Bureau of Standards, and showed that samples No. 1 and 2 (of Table 5) contained 30 to 40 percent chalcedony; Nos. 3 and 4, 10 to 20 per cent; and No. 5, less than 10 per cent.

†Oral communication.

It was hoped at the inception of this study that the heating of samples of rocks under pressure could be used as an accelerated test to determine their fitness for use with high-alkali cement in concrete. The test, however, failed to show the reactivity of siliceous magnesian limestone and Buck phyllite, but did show an extreme reactivity in chalcedony, which has not been found in service. It appears, therefore, that this test should be used only in conjunction with other tests, perhaps of the type suggested by Parsons and Insley [3], or Hornibrook, Insley and Schuman [7].

It should be emphasized that although reactivity of aggregate and expansion of concrete are related, they are not synonymous. Actual tests of expansion in concrete, using suspected aggregate and cement, are undoubtedly the surest method. Since expansion tests require months or years, it is hoped that data obtained by means of the accelerated tests described, will aid in attacking the problem.

5. SUMMARY

The 12 aggregate materials studied most intensively were found reactive to alkalies in the order: opal, chalcedony, pitchstone, rhyolite, basalt, magnesite, obsidian, calcite, limestone, dolomite, microcline, and oligoclase. The order differs slightly with NaOH and with KOH. The carbonates lost more weight in KOH than in NaOH; basalt, microcline, obsidian, oligoclase, pitchstone, and rhyolite lost more weight in NaOH.

A few additional materials studied in one series of tests were found reactive with NaOH in the following order: opaline chert, Potomac chert, magnesite, obsidian, phyllite, and quartz. The behavior of siliceous magnesian limestone was exceptional in that it gained slightly in weight in the tests.

Addition of alkali carbonates to alkali hydroxide solutions did not appear to increase attack on the group of 12 aggregate materials. Alkali sulfates had little effect at 122 C.

The pressure cooker test method throws much light on alkaline reactivity of aggregates and, on the whole, the results of the method agree with alkali etching tests and with service records as reported in the literature.

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Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1945 for publication in the JOURNAL for June 1945.

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Discussion of a paper by Leonard Bean and J. J. Tregoning:

Reactivity of Aggregate Constituents in Alkaline Solutions*

By BRYANT MATHER†

The purpose of the following discussion is to direct attention to certain relationships between the materials opal, chalcedony, and chert; and to suggest how these relationships may elucidate the reported behavior of these materials in alkali-aggregate reactivity.

Throughout the paper under discussion, opal, chalcedony, and chert were reported to have behaved in a manner differing considerably from that of all the other materials tested. From their discussion it appears that the authors regard the reactivity of chalcedony and chert as surprising. They suggest that the reactivity of the Potomac River chert appeared to be due to the presence of chalcedony, since the reactivity of the chert samples was in the same order as the amount of chalcedony present.

Contributors to the discussion of alkali-aggregate reactivity agree generally that opal is the most reactive material yet encountered. Blanks (1)‡ has commented that very small amounts of opal appear to be sufficient to impart activity to an aggregate in the presence of a high-alkali cement. In view of the well-known bad record of opal, there are set forth below certain data regarding that material and the interrelationships of the materials opal, chalcedony, and chert.

Opal has been generally considered to be an amorphous material, even though, at room temperature, its powder has given the x-ray picture of a high-temperature cristobalite (2). In any case, such evidences of submicroscopic crystallinity as have been found suggest affinities with a crystalline form of silica other than quartz. The water content of opal has been reported (3,4) to vary from 2 to 13 per cent, and, as the water

*ACI Journal, Sept. 1944, *Proceedings* V. 41, pp. 37.
†Engineer, War Dept., Corps of Engineers, Central Concrete Laboratory, Mount Vernon, N.Y.
‡See references at the end of this discussion.

content varies, the density varies from 1.9 to 2.3 (5,6), and the index of refraction from 1.406 to 1.460 (4,5).

Chalcedony has been defined as a fibrous, microcrystalline form of silica which is formed through the crystallization of colloidal silica (7). It is supposed to be specifically distinct from quartz since it does not invert at 575 C., yet x-rays reveal the same crystal structure as quartz, while both the index of refraction and the density are lower than quartz (4). The density of chalcedony varies from 2.55 to 2.63 and the principal indices of refraction are n_o = 1.537 (1.533 to 1.539) and n_e = 1.530 (4). The fibrous structure, density, and optical properties of chalcedony have been investigated by Donnay and by Correns and Nagelschmidt. Donnay reports (5) that the fibers of chalcedony are composed of low-temperature quartz and a continuous interstitial material which is probably opal. Correns and Nagelschmidt (8) established by x-ray study that the *c* axes of the constituent quartz crystals, which in large part make up the fibers of chalcedony, are always oriented normal to the elongation of the fiber, and they state that the fibers clearly ("offenbar") contain opal. Donnay (5) shows that the percentage of the interstitial opal, among the quartz crystals which make up the bulk of the fibers of chalcedony, may be calculated from the departure of the measured values for density and indices of refraction from those of quartz. Donnay calculates that the material studied by Correns and Nagelschmidt contains from 8 to 15 per cent opal. In this connection it should be mentioned that the individual particles composing the chalcedony fibers are of sub-microscopic size; whether they are small in comparison to the wave length of light, as hypothecated by Correns and Nagelschmidt, is however open to question. Donnay concludes (9) that if opal is bad, then chalcedony is bad too, on account of its opal content.

The data cited serve to elucidate not only the behavior of opal and chalcedony in the tests reported by the authors but also shed some light on the discussion offered by Rhoades (10) in which he suggested that: "Typical chalcedony perhaps is no more to be suspected of potential reactivity than quartz. However, authoritative conjecture surrounds the possibility that some chalcedony verges toward an opaline character in becoming hydrous, and the question naturally arises as to the possibility of such types also assuming other similarities to opaline silica, perhaps even becoming potentially reactive in alkaline environments. Confusion surrounds this issue, partly because the chalcedonic material is frequently so fine-grained as to preclude satisfactory microscopic study; it is possible that opaline material minutely disseminated in chalcedonic aggregates may escape observation and yet impart opaline properties which appear to pertain to the enclosing chalcedony. This point impinges directly on the problem of the potential reactivity of cherts. . . "

Of all the terms borrowed from petrography for use in discussions of aggregates, chert is perhaps the most difficult to define adequately. Wuerpel and Rexford (11) give an authoritative collection of definitions and descriptions of chert. The material studied by them was cryptocrystalline and composed principally of quartz. They report that opaline silica did not occur in any of their samples in sufficient quantity to be identified. Rogers and Kerr (12), however, regard chalcedony as the principal constituent of cherts. Twenhofel (13) says that while the silica in Mesozoic and older cherts is crystallized and consists of a mosaic of quartz and chalcedony, some of the younger cherts contain opal. The California cherts that have been found to be reactive, and that are known to contain opal, are younger than the Mesozoic. Trask and Patnode (14) studied 25 samples from the Miocene "Monterey" chert formation in California and found them to be separable into two groups: banded or layered chert, and opaline concretions.

The material known as chert, therefore, includes some samples that are predominantly quartz, some that are predominantly chalcedony and thus contain up to 15 per cent of submicroscopic interstitial opal, and some that are predominantly opal.

The writer agrees with the suggestion of the authors that the reactivity of the Potomac River chert is a function of its chalcedony content. He would however go further and suggest that the potential reactivity of the mineral, chalcedony, or of any of the forms of the rock, chert, is a function of the amount of opal contained.

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Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1944 to June 1945.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Mix for Pre-stressed Concrete Tanks Using Coral and Porous Limestone (41-154)

By JOHN G. DEMPSEY*

The location of the construction of pre-stressed concrete tanks necessitated use of local materials for concrete aggregates all of which were derived from coral and shells blown into place from their ancient sea bottom by the winds of the glacial epochs and cemented together by percolating water. They possessed extremely high surface areas and were very variable in quality. Even the best material contained a high proportion of porous or soft material.

A hot semi-tropical sun and the necessity of using sea-water for mixing shortened substantially the interval before initial set, and to offset this and reduce the water-cement ratio a commercial pozzolanic admixture was used. This brought setting time back to normal and permitted lowering the amount of mixing water to five gallons per sack of cement in spite of the high internal friction of the mix.

As the tank concrete would require the maximum strength and density of which the aggregate was capable, great care was exercised to secure

*ACI Member who reports experience in work for the U. S. Navy while he was in charge of the concrete laboratory for the contractor.

accurate proportions in the batches sent out in addition to close control of water content and placement methods. As a result of this care and the pains taken by the job foreman and the Navy inspectors the finished surfaces of floor and walls were very hard and dense and seven-day compressive strength exceeded 4,000 psi.

The mix decided on was 1:1.92:2.25 by weight. Previous experience had proved the desirability of a high mortar content because of the very high surface area of even the smallest particles. Further tests of the aggregate had shown the following characteristics:

	Fine Aggregate	*Coarse Aggregate*
Fineness modulus	1.07	6.45
Absorption, percent	4.4	8.1
Specific gravity *	2.54	2.22

*Apparent specific gravity, saturated surface-dry basis. Three consecutive batch control tests made at the plant discharge hoppers showed the following conditions:

	1	2	3
Percent surface water on sand	3.4	3.4	3.4
Percent sand retained, No. 4 sieve	25.7	17.6	18.1
Percent stone passing No. 4 sieve	24.0	6.8	4.0

Following each control test that showed more than two per cent variation from its predecessor, field proportions were computed according to the method outlined, p. 146, Bureau of Reclamation's Concrete Manual. Samples from the hoppers, however, were run through the No. 4 sieve in their field condition rather than after drying to diminish the lag between test and adjustment of the batching scales. The computation of the second test is shown here: Mix proportions by weight were 1:1.92:2.25 and the charge of cement was 2,000 lb. plus 21 lb. of admix.

Wt. of dry sand desired, $2{,}000 \times 1.92 = 3{,}840$ lb.
Wt. of damp sand, $3{,}840 \times 1.034 = 3{,}971$ lb.
Wt. of stone desired, $2{,}000 \times 2.25 = 4{,}500$ lb.

	First Trial		*Second Trial*	
Sand sizes	6.8 per cent × 4500 = 306		6.8 per cent × 3988 = 271	
	3971 − 306	= 3665	3971 − 271	= 3700
	3665 ÷ 82.4 per cent	= 4448	3700 ÷ 82.4 per cent	= 4490
	17.6 per cent × 4448 = 783		17.6 per cent × 4490 = 790	
Stone ¾ in. max.	4500 − 783	= 3717	4500 − 790	= 3710
	3717 ÷ 93.2 per cent	= 3988	3710 ÷ 93.2 per cent	= 3981
	(−No. 4) + (+No. 4) desired,		3971 + 4500	= 8471 lb.
Check	First trial		4448 + 3988	= 8436
	Second trial		4490 + 3981	= 8471 chk

Accordingly, from this computation and the usual adjustment of water,

Set scales at { Sand 4,490; Stone 3,981; Water 779 } to secure { Minus No. 4 3,840 lb.; Plus No. 4 sizes 4,500; Water 910 }

By running the separation test through the No. 4 sieve in the damp field condition much closer control could be exercised. Earlier tests established the relationship between the dry and damp sieving. All batch control computations were made by slide rule. The surface moisture tests were made with the Chapman flask.

Test cylinder data for this pour are shown at the end of these notes, together with similar data from a pour typical of the mix used for most of the well over 50,000 cu. yd. poured in this location. The pozzolanic admixture was used generally to improve workability and to save cement. Through its use specification strengths were secured at a saving of one sack per cubic yard. On the tanks, however, it was used to reduce water content and increase density. Average job concrete had a rather soft, chalky surface in great contrast to the hard, glassy finish secured on the tanks.

Data on Test Cylinders

	Tank Mix	*Standard Mix*
Specimen Number	123	78
Mix proportions by weight	1:1.92:2.25	1:2.24:2.64
Water-cement ratio by weight	0.455	0.513
Cement factor, sacks per cu. yd.	7.0	6.1
Pozzolanic admix, lbs. per sack	1	1
Sand-aggregate ratio by wt.	46 per cent	46 per cent
Slump	3½ in.	5 in.
7-day compressive strength	4,068 psi	3,540 psi
28-day compressive strength	4,610 psi	4,245 psi
Type of break	straight split	cone shear

Durability of Asbestos-Cement Siding (41-155)

Q—We are contemplating the use of corrugated asbestos cement siding and roofing for some buildings, and are searching for data on the useful life of the material. Does exposure destroy the cement bond in a few years? Is coating recommended; if so, what type; what is the reasonable life expectancy: without coating? with coating?

By C. R. HUTCHCROFT*

A—Exposure does not destroy the cement bond in corrugated asbestos-cement siding over an extended period of years.

We do not particularly recommend coatings, but we do under certain conditions, recommend asphaltic impregnation of the asbestos cement corrugated—at our plant—to enable it to resist certain exterior influences, such as: (a) acid fumes, and (b) extreme water condensation. By far the great majority of our jobs are not treated in any way, as such conditions exist only in limited areas—of chemical plants usually.

The reasonable life expectancy—without coating—depends entirely upon the environment under which the asbestos sheet is exposed. Under normal atmospheric conditions we do not know how long the asbestos cement may last as we have been manufacturing such material only since 1904, and failures on jobs put up at that time have not occurred.

Reference is made to the Lakehurst Naval Air Station Hangar at Lakehurst, N. J., a very large structure erected in 1919 of asbestos cement

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corrugated. It is believed that the Baldwin Locomotive Works in Philadelphia has a large area covered with early material. The American Viscose Co. in Nitro, W. Va., applied large areas of corrugated. During recent years there has been very extensive use of this material by many government agencies: for Naval Supply Depots, Army Air Service Hangars, and other similar jobs. The Dow Chemical Co. has used very large quantities of it in the construction of magnesium plants.

The reasonable life expectancy with coating depends entirely upon the environment. We have had corrugated in areas where acid fumes condensed on the undersurface and caused failure in two to three years. Application of treated material rectified the condition and such structures are still in satisfactory use after a longer period. We ask the customer how and where he proposes to apply the material, and under what temperature, atmospheric and vibration conditions, and recommend treated or untreated material applied in one of three ways. There are some places where we do not recommend the use of corrugated at all.

Frozen Concrete Cheaply Repaired (41-156)

By WALTER H. WHEELER*

Section A-A, Fig. 1, shows a half elevation of the main girder and Section B-B a cross section of the girder and part of the balcony and second floor of an auditorium. The girder is supported at its center on a steel pipe column. The total live and dead load on this column is approximately 340 kips.

Concrete for the front half of the balcony including the main girder was placed when the temperature was approximately 10 F. The mixing water and aggregates were not heated. It is said that high early strength cement was used. When the writer first saw the work a small area of the side forms of the girder had been stripped, exposing a void in the bottom of the girder, immediately above the pipe column, about 5 ft. long by about 1 ft. high and extending through the girder. There was evidence of frost in the girder and on the top of the seat deck. Centering to support a 100 ft. span concrete roof girder had been erected on top of the balcony girder, but the roof had not been poured. The writer left instructions with the supervising engineer to have all side forms removed from the girder and balcony beams and face of the seat deck, but to leave all vertical supports in place and to notify him when this had been done. He also directed that salamanders be placed under the balcony and kept fired, and that an air compressor and drills and a crew of men be on the job, when he returned, to work under his direction. He also directed that means be provided at once for heating mixing water

*Minneapolis, Minn.

and aggregates before any more concrete was placed in the work, and to do nothing with the balcony or roof above balcony until he had examined the work and a decision was reached.

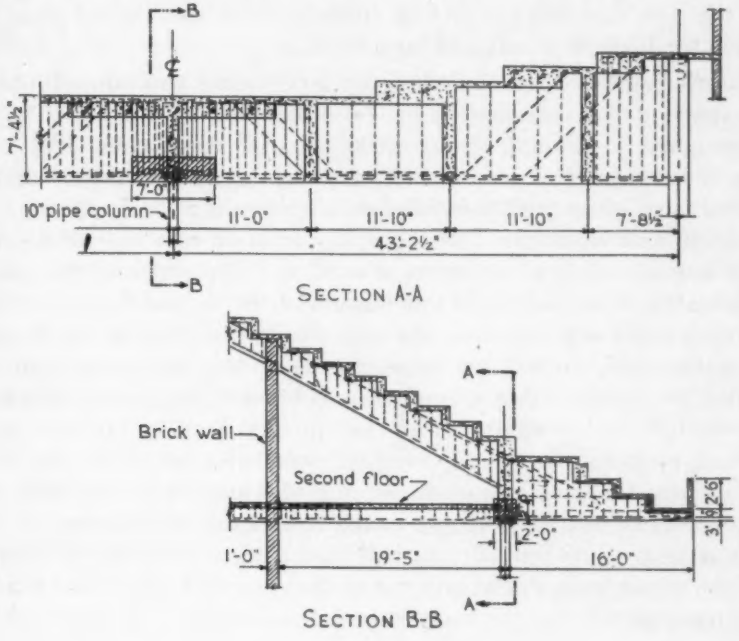

Fig. 1

The structural drawings showed a steel cap, 20 in. by 20 in. by 2 in. thick, welded to the top of the pipe column with 4 anchor bolts 1¼ in. diameter by 30 in. long extending up into a 24 inch square post integral with the girder. This post and the anchor bolts had been omitted in building the girder.

After the forms were stripped, the writer found that except for three or four rough spots in the main girder and cantilever beams, where bars were exposed, and ¼ in. to ½ in. of soft concrete on top of the seat deck, the forms seemed to have been well filled and the concrete seemed to be sound and hard. Holes were drlled through the main girder at intervals of about 10 ft. for its entire length, which showed the concrete to be sound, dense and hard. It was therefore decided to leave the work in place and repair it.

A section of the concrete in the girder over the pipe column, 7 ft. long and 2 ft. high was cut out and a notch cut in the cantilever beam and in the second floor slab. These cuts were made by drilling continuous

lines of holes, and then breaking out the remaining concrete with jack hammers. This left the faces of the concrete square to receive the new concrete. After the old concrete was removed, the exposed bars in the bottom of the girder, which had been poorly placed, were pried apart so that the new concrete could flow freely around and embed each bar. The anchor bolts were set, and form built.

A new concrete section 7 ft. long by 2 ft. square was poured into the open space. This is indicated by cross hatching on Fig. 1. The mix was designed to develop about 4,000 psi in 7 days, using high early strength cement. The mixing water and aggregates were heated and the concrete kept warm with salamanders after pouring. The concrete was mixed stiff and vibrated. The top of the form on each side of the main girder was left off until the form was filled. The concrete was poured and vibrated through the cut out portion of the second floor slab, while the writer and the supervising engineer stood on staging on the opposite side of the main girder, one on each side of the cantilever beam and watched the concrete flow around the steel and fill the form. When the form was full, the top was put on and two prismatic spouts 4 ft. high, which had been previously prepared, were set over holes left in the top of the form, against the back side of the main girder and these were filled with wet concrete to put the concrete in the new patch under pressure while it was setting. Sufficient shoring had been placed under the main girder while the repair work was in progress so that it would safely take the load of the main girder plus the load from roof centering.

This work required 2 nights and 3 days to complete, working double shift. The forms were removed the next morning and the patch found to be very dense and hard and completely filled. Pouring of the roof girders proceeded immediately.

Instructions were given to remove the soft top concrete from the seat deck and to finish it with a 1 inch topping of portland cement mortar, also to patch up the other rough spots in the main girder and balcony beams.

At the time the repairs were made the engineers in charge of the work proposed that a load test be made on the balcony before it was placed in service, to which the writer agreed. However, they later decided that this would not be necessary. The job has been in use at this writing for more than a year.

As there is a suspended metal lath and plaster ceiling under the balcony, which forms the plenum chamber for ventilation, the patch is concealed in the finished job.

Workmen on the job, told the writer, that when the main girder was poured, the concrete stiffened so rapidly in the low temperature that it

was practically impossible to vibrate it effectively. Under all the circumstances, it is fortunate for all concerned that it turned out as well as it apparently did.

Color Variation in Concrete Products (41-157)

Q—A products manufacturer in the Philadelphia region has submitted the following inquiry to the Technical Advisory Service of the Smaller War Plants Corporation: When we mix our concrete, the water content in it determines the color of the finished product. The concrete we use cannot be poured but is of a dry nature and this concrete is vibrated in the molds. It is impossible for us to keep the water content the same in each batch because as the material is delivered, it varies from a very wet load to a very dry load, and we have no mechanical means of determining the wetness of the concrete mix. If we had such a device, we would know whether to add water to the mix or not to add water. In other words, we are looking for some mechanical device or means of determining the water content of concrete.

By J. W. KELLY*

A—The most obvious remedy is to remove the cause—that of variation of the material (presumably the aggregate) "from a very wet load to a very dry load". Situations of this kind can be corrected by insisting on reasonably uniform moisture content of the aggregate, or at least that the aggregate from either wet or dry sources be furnished for as long a period as possible before changing to the other source. Prevention of variation is doubly important if the aggregate is measured by volume, as variations in moisture content of sand cause variations in its bulking and therefore in the actual number of sand grains measured into the batches.

Assuming, however, that some variation in aggregate moisture is unavoidable, the amount of this moisture could be determined and then the remaining water added to bring the total water content up to the desired amount. One quick moisture test is to dry out a small sample with heat, using a sensitive scale to determine the loss in weight. Several displacement methods have also been used, usually involving the filling of a container to a given mark with water, then adding a weighed quantity of the damp sand, then reading the increased volume which is translated into moisture content of sand by means of a formula or a diagram. The standard test of this nature is the American Society for Testing Materials Designation: 70-30, "Standard Method of Test for Surface Moisture in Fine Aggregate," which is to be found in the 1942 A. S. T. M. Standards, Part II, p. 377.

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It is believed that control of the measured amount and of the moisture content of the aggregate will control the water content of the concrete better than any test of dry products concrete which has yet been attempted. The common rough test for consistency of this kind of concrete is to squeeze a handful of it from the mixer and to "judge" the water content. Again, the water control is usually accomplished more closely by watching the performance of the first units in the molds and adjusting the water content to secure the wettest possible consistency without "slumping" of the unit upon stripping. If however, the actual *water content* of the concrete is desired, it can be obtained by drying out a sample or perhaps by a displacement method paralleling that for moisture content of sand alone; in the latter case the container could be calibrated by introducing samples of concrete of known moisture content—that is by adding known amounts of water to the dry materials.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Simplified charts for double-reinforced beams

DONALD L. TODD, *Engineering News-Record*, V. 132, No. 26 (June 29, 1944) pp. 98-99.
Reviewed by S. J. CHAMBERLIN

The article discusses the construction of simplified design charts for the selection of double-reinforced beams. Three charts, based on the ACI 318-41 code, for 2000, 2500 and 3000-psi concrete are illustrated. An example in the use of the charts is included.

The torsional resistance of plastic materials with special reference to concrete

W. T. MARSHALL, *Concrete and Constructional Engineering*, V. 39, No. 4 (Apr. 1944) pp. 83-88.
Reviewed by GLENN MURPHY

Theoretical values of maxium torque developed in members of circular and rectangular cross section composed of an ideal plastic are compared with test results recorded in the literature. From the results it appears that the ultimate torsional moment of resistance may be obtained by taking the maxium stress equal to the tensile strength of the material.

The development of pre-stressed concrete

KURT LENK. *Beton u. Stahlbetonbau*, 42:145, Oct. 15, 1943. Reviewed by A. U. TEUER

This is a general review of the history and development of prestressed concrete. The author points out in detail the different field applications of prestressing that have been made to date. He concludes that prestressing has eminently proved its worth in recent years in the manufacture of pipes and precast beams, and that with time and experience, economically feasible means of application to in situ construction will be found.

Soil-cement used for roadway maintenance

L. E. AKERS, *Engineering News-Record*, V. 132, No. 26 (June 29, 1944) pp. 92-93.
Reviewed by S. J. CHAMBERLIN

The Virginia Department of Highways has used considerable quantities of soil-cement for patch work in secondary roads consisting of stabilized paving and for emergency repairs of higher types of pavement. Twelve percent of cement is added to the selected soil with water at a central plant and the mixture is delivered by truck. Broken pavement and unstable subsoil is removed, crushed stone backfill is added and the mixture is compacted in 4-in. layers with pneumatic tampers. Patches made with soil-cement mixtures containing less than nine percent moisture are opened immediately to traffic.

Specifications for concrete railway sleepers

Concrete and Constructional Engineering, V. 39, No. 5 (May 1944), pp. 131-134.
Reviewed by GLENN MURPHY

Report of a revision of the British Standard Specification for Concrete Railway Sleepers was published as B.S. 986/1944 by the British Standards Institution.

Proportions by volume are specified. A slump of 1 inch and a minimum 28-day standard cube strength of 6000 psi for pre-stressed concrete and 5000 psi for ordinary concrete are stipulated. Design loads are given and a minimum resisting moment of 30,000 in. lb. is specified. A maximum compression stress of 1200 psi is established for the concrete and 18000 psi for the steel. Additional specifications are given for pre-stressed concrete, and for fabrication.

Specification for twisted reinforcement bars

Concrete and Constructional Engineering, Vol. xxxix No. 2 (Feb. 1944) p. 45.
Reviewed by GLENN MURPHY

The British Standards Institution has published a specification (B.S. No. 144, 1943) for Cold Twisted Steel Bars for Concrete Reinforcement. It applies to twisted bars and twin-twisted bars. The area to be used is the area of one or two bars before twisting. The pitch of a complete twist is between 8 and 14 times the diameter or thickness, and the bars must conform to B.S.S. No. 785 before twisting. Values of minimum ultimate strengths ranging from 63000 psi to 80000 psi and minimum yield stresses from 54000 psi to 70000 psi are stipulated. Both cold bend tests and minimum elongation requirements are included as a check on ductility.

Nomograph for computing sections of certain reinforced concrete beams

G. JAUSON, *LeGenie Civil*, V. cxx, No. 11, June 1, 1943, pp. 125. Reviewed by R. L. BERTIN

The nomograph is so drawn as to permit solving any two of the following four variables in terms of the others:

Bending Moment
Tensile Steel Stress
Compression Concrete Stress
Percentage of Tensile Steel

It is limited to rectangular sections without compression steel reinforcement subjected to bending only.

Righting a 24-story tilted building

Concrete and Constructional Engineering, V. 39, No. 4 (Apr. 1944) pp. 99
Reviewed by GLENN MURPHY

A reinforced-concrete frame building 300 ft. high in the city of Sao Paulo, Brazil, that leaned more than 2 ft. out of plumb, has been righted by hydraulic jacking after freezing the subsoil and installing deep concrete piers. After unsuccessful attempts to halt settlement by cement grouting and by injection of an aluminum salt to coagulate the lense of sand responsible for settlement, doublewalled circulation pipes were installed to a depth of 60 ft. below the lense. Brine was circulated through the pipes lowering the temperature of the soil to -20 C. Following which holes were excavated and filled with concrete (accelerated with calcium chloride) to form piers. The building was righted with hydraulic jacks placed on the piers.

The resistance of concrete to frost

Concrete and Constructional Engineering, V. 39, No. 4 (Apr. 1944), pp. 100-107.
Reviewed by GLENN MURPHY

Extract of a paper in *Journal of the Society of Chemical Industry* by A. R. Collins, containing a summary of investigation conducted by the Road Research Laboratory

of the Department of Scientific and Industrial Research. Causes and types of disintegration are discussed. It is felt that probably liability to damage is a result of high porosity combined with small pores. Freezing and thawing tests (-14 to +25 C.) on a daily cycle were conducted on 4-inch cubes of concrete of various water-cement ratios. The resistance to disintegration as measured by the crushing strength was appreciably greater for the concrete of low w/c. Supplementary tests indicate that the water-cement ratio is the principal factor in determining frost resistance of concrete.

Use of twin bridges simplifies design

Engineering News Record, V. 133, No. 2 (July 13, 1944), pp. 106-108. Reviewed by S. J. Chamberlin

Twin bridges of identical lay-out for a dual-lane express highway consist of 160-ft. reinforced-concrete, open-spandrel center arches flanked at both sides by 100-ft. arches and girder approach spans. For each bridge two lines of arches are used in the main spans and three lines of girders in the approach spans. The 160-ft. arch ribs are six ft. wide and vary in depth from 4 ft. 9 in. to 2 ft. 6 in. The ribs of the 100-ft. arches are six ft. by 3 ft. 1 in. to 1 ft. 8 in. Girders are all 21 in. wide and vary in depth below the 8-in. deck from 4 ft. 1 in. at the center to about 7 ft. over the open bents. The gap between the two bridges is closed by a reinforced slab on a 20-in. span which forms the central portion of a raised median strip 6 feet wide.

Precast arch ribs for an auditorium roof

William L. Zeigler, *Engineering News Record*, V. 133, No. 2 (July 13, 1944) pp. 89-91.
Reviewed by S. J. Chamberlin

Parabolic-shaped, three-hinged precast ribs of 9½ by 19-in. cross-section were used on a 90-ft. span with a 25-ft. rise. The ribs were cast in timber forms laid out on a work platform. A 10-in. pipe was placed in the forms for the crown hinge, the reinforcing for one half of the rib being welded to the pipe and the other half merely cast against the greased pipe. In erected position the half ribs are connected by a pair of steel plates bolted to the one rib and engaging the ends of the pipe in a loose fit. The lower ends of the ribs are rounded and greased to fit in circular bearings in precast seat blocks. The ends of the ribs were cast against one side of a 16-gage steel plate bent to radius and the seats cast against the other side of the plate. The arches were placed by crawler cranes, the steel plates slipped over the protruding pipe on one half and then bolted to the other half. Cast in place struts were built 7 ft. above the floor to connect the ribs.

Some properties of concrete under sustained combined stresses

C. Martin Duke and Harmer E. Davis—Presented at the 47th Annual Meeting of the American Society for Testing Materials, June 26-30, 1944.

The paper presents the results of laboratory investigations on the creep of concrete under combined stresses. Most of the tests were conducted on 8 by 16-in. cylinders, using three different ratios of axial to lateral stress. In addition, some tubular specimens were tested in torsion. Supplementary tests were made to study the effect of pore pressure on strength of concrete under combined stresses.

A method is tentatively advanced for estimating the approximate magnitude of creep under combined stresses from the results of measurements of creep under simple axial compression. When applied to the results of the tests herein presented, the method gives estimated creeps which are reasonably close to the measured creeps during the first few weeks after loading. It is thought that changes in cement gel structure induced by the applied water pressure may be responsible for differences in estimated and measured creeps at later times.

A million-gallon elevated tank and reservoir

Concrete and Constructional Engineering, Vol. xxxix No. 2 (Feb. 1944)

Reviewed by GLENN MURPHY

Reinforced concrete was used for the construction of a 1½ million gallon reservoir and a million gallon capacity elevated tank 37 ft. 6 in. above the reservoir. The structure is rectangular in plan—165 ft. by 85 ft. The tank is supported on columns about 16 ft. o.c. running up through the reservoir. An 11-in. slab was used for part of the tank floor and a 9-in. slab for the center portion. Vee-grooves at the inside boundaries of all construction joints were carefully sealed with a bituminous material followed with two coats of a bitumastic solution. Special precautions were taken to prevent leakage at the junction of the walls and floor by thoroughly roughening and cleaning the concrete floor, and applying a layer of mortar, followed immediately by the concrete for the walls. The structure constitutes one of the largest, if not the largest, storage tanks in the world.

Curved beams with constant section and parallel fibres

J. FERRANDON, *LeGenie Civil*, V. cxx, No. 13, July 1, 1943, pp. 149, 150. Reviewed by R. L. BERTIN

The study has two objects in view:

1. To define a method of generating such beams
2. To examine the application of the fundamental principles of the theory of flexure to their solution

The beam is considered generated by the displacement of a plane surface of fixed shape and dimensions along a curved directrix so that the axis of the surface follows the directrix and the surface remains constantly normal to it.

The detailed mathematical analysis given leads the author to conclude that such beams can be analyzed by means of the conventional principles of flexure modified by some functions dependent upon the geometrical characteristics of the neutral axis.

Study of the continuity of arches

HENRI BORDIER, *LeGenie Civil*, V. cxx, No. 18, June 15, 1943, pp. 138 and 139.

Reviewed by R. L. BERTIN

Consideration is given to a system of continuous arches of varying spans and sections resting freely on non-yielding supports.

One of the spans is loaded with any system of loading, and the study is devoted to first finding the support reactions, then, the bending moments, vertical reactions, and thrust within that span and then the distributed moments at the other supports as well as the corresponding reactions by means of equations derived by means of the moment area method.

These equations establish the fixed points and carry over factors in terms of the dimensional characteristics of the arches with which the support moments for any kind of loading can be determined and distributed along the entire system.

A pioneer engine test building

Engineering News-Record, V. 132, No. 24 (June 15, 1944) pp. 89-91 Reviewed by S. J. CHAMBERLIN

The heavily reinforced concrete building consists of a large central portion with three pairs of test cells 95 ft. long on each side. The major problem was the interior forms for the cells which varied from 20-ft. diameter at the throat to 20 ft. square at the ends. The form for the 6-ft. long uniform-diameter section was constructed lying flat on a work platform then set on edge. Next, the tapered end sections, constructed while resting on their large ends, were up-ended and joined to the cylindrical section. Finally, the entire form for the throat, now 38 ft. long was picked up by a crawler crane and set in place. The floor and roof slabs are 18 in. thick, while the walls vary in thick-

ness from 18 to 24 in. In constructing a pair of cells the floors and the lower half of the two outer walls and the intermediate wall were placed in one operation, next the upper half of the walls and finally the roof. Vertical joints were at 60-ft. centers. To reduce vibration, foundations were carried to rock and footings for engine test stands were isolated from the rest of the structure.

Elastic, plastic and permanent deformations of several types of concrete

J. Bolomey. Bull. tech. Syisse romande 68, 169-73 (1942); Chem. Zentr. 1942, II, 2307. Chemical Abstracts, V. 38, No. 10, May 20, 1944. Highway Research Abstracts

Specimens of concrete which were produced by various processes and which had been kept moist for varying periods were kept for 800 days under loads of 0, 710 and 1420 psi. From the deformations so produced the following conclusions were drawn: The deformation (elastic, plastic and shrinkage) were proportional to the load and were less the longer the preliminary hardening in water had been continued. They were closely related to the density of the concrete; with the same load, the deformations were greater for soft concrete than for concrete which was only moist (as damp earth) and they were less the higher the modulus of elasticity of the concrete. No proportionality was found to exist between the deformation and the resistance to compression. On the other hand, the deformations under continuous load, which did not completely cease even after more than 2 years, were approximately proportional to the shrinkage, with the deformation under a load of 710 psi being about twice, and that under a load of 1420 psi being about 3 times, that observed under no load.

Comparative tests of 52 brands of portland cement from 15 countries

Eugene V. Barrett—Presented at 47th Annual Meeting of the American Society for Testing Materials June 26-30, 1944.

This paper presents the results of comparative chemical and physical tests made on 52 brands of portland cement from 15 countries and includes the results of tests of 90 samples of one of these brands of cement that contained from 5.5 per cent to 9.6 per cent magnesium oxide (MgO). The investigation was started early in 1938, and cement was tested from Yugoslavia, Belgium, Germany, United States, Holland, France, England, Japan, Venezuela, Norway, Poland, Spain, China, Denmark, and Sweden.

The tests included tension tests on 7- and 28-day briquets, fineness, time of set, steam soundness, autoclave expansion, chemical analysis, and compression tests on 6 by 12-in. concrete cylinders made so that the cement was the only variable. The test methods were for the most part those recommended by the A.S.T.M.

Test results indicate that the expansion in the autoclave test does not bear any relation to the total MgO content of a cement. The 7-day briquet strengths bear some relation to constant water-cement ratio concrete cylinder strengths but cannot be considered as a reliable index of cylinder strength. The 28-day briquet strengths bear no relation to cylinder strengths. Most of the 52 brands of portland cement tested complied with A.S.T.M. specifications effective at the time the cements were tested, and several countries produced excellent quality portland cement.

The effects of certain variations in consistency and curing conditions on the compressive strengths of cement-lime mortars

G. J. Fink—Presented at 47th Annual Meeting of the American Society for Testing Materials, 260 S. Broad St., Philadelphia 2, Pa. June 26-30, 1944. Author's Synopsis

Compressive strength data are presented on a series of cement-lime-sand mortars in which the ratio of cement to lime ranged from 1:1/4 to 0:1, by volume, and in which the ratio of cementitious material (cement plus lime) to standard Ottawa sand was 1:3, by weight. The results show that, with two exceptions, the 28-day strengths of the

cement-lime mortars of 75 per cent flow were higher than those of the mortars of 110 per cent flow, the differences increasing with the increasing cement content. The reverse was true in the case of the straight lime mortars.

Data are also presented showing the effects of different methods of curing on the compressive strengths of specimens of mortars. The results indicate that the strengths of the mortars containing the highest proportion of cement (1:1/4:3 by volume) were practically the same when the test specimens were cured for 28 days in the damp closet as when the specimens were cured for 7 days in the damp closet and 21 days in water, but that in the case of the mortars with lower ratios of cement to lime, the specimens cured for the full period in the damp closet had significantly higher strengths than those cured under the other condition. Curing for 7 days in the damp closet, 18 days in water, and 3 days in laboratory air produced specimens with much higher strength than curing by either of the other procedures.

The bending of square footings

A. GROTKAMP. *Bauing.*, 1943, 23 (25/26), 189-94. Building Science Abstracts, V. XVI (New Series) No. 11 (Nov., 1943) (London). HIGHWAY RESEARCH ABSTRACTS

The moment distribution in square foundation slabs of constant and of variable thickness under a centric load is considered, uniformly distributed soil compression being assumed. Design of slabs to withstand bending is examined in relation to the usual methods of design and a new approximation method is prepared. Conclusions reached from the investigation are as follows: The cross-section most subjected to bending is that through the axis of the footing. A series of model tests of unreinforced cement mortar slabs of constant and variable thickness on a foundation of sand has shown that the crack passes through the center of the slab and is parallel to one side. The whole of the earth pressure to the left or right of the section contributes to bending. If the area under load is assumed to be in the form of a trapezium the slab will be subjected to a considerable degree of overstressing. In the case of slabs of constant thickness the width of slab under compression is to be assumed as about one-half, and for those of variable thickness as about one-half to two-thirds of the length of a side. The slab thickness in the record case may be taken as about five-sixths of the greatest thickness at the column base. Attention should be given very especially to the distribution of reinforcement. The bars are to be placed parallel to the side of the slab, the spacing to increase with distance from the axis, so that at the edge the space between the bars is six times that beneath the column. If the bars are uniformly spaced those in the middle part of the slab are subjected to about double the design stress.

Use of pre-cast pre-stressed concrete beams in bridge deck construction

A. A. PAUL, *Journal, Institution of Civil Engineers*, 1943-44, 21 (1), 19-30. Highways, Bridges and Aerodromes, March 22, 1944. (London). HIGHWAY RESEARCH ABSTRACTS

In the reconstruction of certain weak bridges in Great Britain use is being made of pre-stressed reinforced concrete beams, constructed and stored by the Ministry of War Transport as reserve material for emergency bridge construction. The main reinforcement consists of 0.2-in. hard steel wire laid parallel to the length of the beam near its lower edge. The wires are kept under tension during the casting of the beam, and are released at the end of the curing period. Contraction of the wires on being released compresses the concrete along the lower flange, the beam assuming a slightly arched form. Pre-stressed members of this type must be lifted and supported from the ends only and with the convex surface uppermost. Descriptions are given of the reconstruction of the decks of two bridges over railways. The clear skew span of the first bridge (42 ft. 6 in.) would normally require 46-ft. beams, but the use of 44-ft. beams, the most suitable length available, was rendered practicable by placing them at slight

angle to the line of skew, the resulting triangular spaces at the parapets being bridged by concrete slabs. The box-shaped cross-section of the beams permitted the joints to be filled and water-proofed with gunned cement. A reinforced concrete carriageway and two footways were provided. For the second bridge, which had a clear skew span of 45 ft. 10 in., 54-ft. H-section beams were used. Since the shape of the section prevented the effective use of gunned cement, the joints were packed at the bottom with tarred felt, or similar material, and tarred paper was laid over the upper surface. Load distribution was effected entirely by the reinforced concrete road slab.

Defects in concrete surfacings on the motor roads

L. Hirche. *Strassenbau*, 1942, 33 (13/14), 69-73. Road Abstracts (Roads and Road Construction) V. XXI, No. 250, Oct. 1, 1943). Highway Research Abstracts

Although precautions were taken to maintain the highest possible quality of materials and workmanship in the construction of the German motor roads, it was recognized at the outset that some failures were inevitable, and records were kept of the construction procedure and the performance of all slabs on a large number of motor road sections to detect any faults that might arise and to trace whether they occurred through faulty construction or from other causes. This report, presented at a meeting of the Concrete Roads Group of the German Road Research Society, gives the results so far obtained. Although all defects were noted, only cracks were evaluated as indicating the durability of the concrete surfacing. Of the sections examined, 75 per cent contained less than 6 per cent of cracked slabs; 14 per cent contained 6 to 15 per cent; and 11 per cent contained over 15 per cent. The standard can be considered high since in the sections containing less than 6 per cent of cracked slabs the greater part of the cracks were either hair cracks, or could be repaired by forming a joint. In sections containing more than 32 cracked slabs per mile the causes of cracking in 45 per cent of the cases are unknown; cracking has occurred in some slabs in spite of satisfactory workmanship, adherence to specification requirements, and absence of subgrade weakness; subgrade failure caused cracking in 39 per cent of the cases observed; faulty construction was the cause in 14 per cent; and incorrect design in 2 per cent. No relation was found between the type of finishing machine used and the defects in the slabs. The proportion of faults in surfacings constructed in 1934-35 is somewhat higher than in those constructed in 1936-37.

Experience with concrete houses

Editorial Note, *Concrete and Constructional Engineering*, V. 39, No. 5 (May 1944) pp. 109-110.
Reviewed by Glenn Murphy

Comments on a report issued by a committee of the Ministry of Works dealing with the performance of concrete wall construction used in the housing program following the last war. The report indicates that most of the concrete houses are satisfactory, that maintenance costs have not been excessive, and that the occupants are pleased with them. About 1000 houses had 6¾ in. walls cast between two $\frac{3}{16}$-in. asbestos-cement sheets, using a clinker or clinker-gravel aggregate. Subsequent cracking of the asbestos-cement sheets allowed penetration of rain. More than 2000 houses were built with a 4-in. wall of 1:6 ballast concrete cast between permanent forms of 2½-in. clinker-concrete slabs. The inside was plastered. These houses are satisfactory. About 6000 houses were built with walls consisting of two 3 or 3½-in. panels of clinker concrete separated 2 or 3 in. Aside from some failures due to unsound clinker and some small cracks at window openings these have been satisfactory. More than 16000 houses were built using precast concrete posts with an infilling of concrete slabs to form hollow wall construction of various thicknesses. Except where unsound clinker was used, this construction was satisfactory.

Performance of the walls was compared with a standard 11-in. hollow brick wall. The committee reports little difference in initial cost, about the same speed of erection, except for the last type which was erected more rapidly, equal insulating qualities and no greater cost of maintenance. The houses were standard construction except for walls.

The measurement of concrete strength by embedded pull-out bars

BAILEY TREMPER—Presented at the 47th Annual Meeting of the American Society for Testing Materials, June 26-30, 1944.

The difficulties of making an accurate estimate of the strength of concrete in structures during the curing period by means of test cylinders are well known. A principal source of error is the difference in environment between the test cylinder and the actual structure. A group of Russian engineers has experimented briefly with several means of determining the strength of concrete in place. One of the most promising methods consisted in embedding the enlarged ends of iron bars a short distance from the face of the concrete while it was being placed. The force required to pull the bar out of the hardened concrete was used as a measure of strength.

This paper reports the results of a laboratory investigation in which 120 pull-out specimens and 120 cylinders were cast from 30 batches of concrete. Mixes and test ages were varied over usable ranges. Both uncrushed and crushed gravels were used as coarse aggregate. The coefficient of variation of pull-out tests was only slightly greater than that of compressive strength tests. A high degree of correlation exists between the two tests. The data indicate that a relatively simple equation can be applied to results of pull-out tests to obtain the equivalent compressive strength up to values of 3500 psi. The method does not appear to be suitable for concrete of higher strength.

It is contemplated that field tests would be made with a dynamometer or a gage-equipped hydraulic jack. The locations in the structure at which pull-out bars would be installed could be selected with a view to final appearances.

The test has the advantage of giving direct indication of the strength of the concrete in the structure. It is free from the complications which arise when an attempt is made to cure separately-cast specimens in the same manner as the concrete in the structure.

Pressure of concrete against forms

RUDOLF HOFFMAN, *Beton u. Stahlbetonbau*, 42:130, Sept. 15, 1943. Reviewed by A. U. THEUER

The author reviews the work of earlier investigators. The undependability of results obtained through the use of membrane pressure cells in earth pressure studies is stressed. As a practical means of acquiring reliable data that will cover all the variables entering the problem, it is suggested that strain measurements be made in the field on the anchor bolts of actual formwork as the concrete is being placed, and implemented by similar measurements on the anchor bolts of specially constructed test boxes.

Using 4 Maihak strain gages the author describes his own experiences using a wood lined test box 3.38 feet high and 3.28 by 7.00 feet in cross-section. The ends of this box were supported by rigid steel brackets set in concrete, and the sides by pairs of channels. The channels rested on rollers and were secured laterally by 4 anchor bolts with one bolt at each end of corresponding pairs. In the experiment the box was filled in three equal lifts, each lift being vibrated into place. Superimposed loads were then placed on the concrete in five equal increments and at such times as to maintain an average rate of load application equivalent to 1.16 ft. per hour rise of concrete in the forms. The superimposed loads were considered equivalent to an equal weight of concrete placed by vibration methods. The large increase in lateral pressure following vibration (over 100 per cent in the case of the second lift) led to the assumption that

for the given width-height ratio of the text box, a reduction of vertical pressure in the mass of the concrete due to side wall friction was a negligible factor.

The maximum lateral pressure was observed immediately after application of the second superimposed loading, nine hours after the start of test. An increase in the lateral pressure was noted for all subsequent loads, however, the time decrease was more than sufficient to offset these increases. By manipulation of the bolts, the effect of small form movements in decreasing the lateral pressures was demonstrated. Discussion of the effect of height-width ratios of formwork on lateral pressure follows.

The development of the lateral pressure formulas of Noack and Boehm are discussed at some length. The author states a preference for an exponential form of expression for the ratio of conjugate stresses and suggests the following:

$$\lambda = \lambda_o e^{-a\left(t-\frac{x}{v}\right)}$$

where λ = the coefficient of conjugate stresses at time t, and height x above the bottom of the box.

λ_o = the value of λ at time, $t=0$.

S = the lateral pressure.

a = a coefficient of dimension s^{-1}, and a measure of the time rate of decrease of s.

λ is considered to be dependent on the nature of the aggregate, water content, and density of the mix, and a, on the nature of the cement and the temperature. Multiplying the foregoing expression by the following

$$p = \gamma\,(vt-x)$$

where γ = the specific weight of the mix and

v = the rate of rise of the concrete in the forms, the author arrives at the following expression for the lateral pressure:

$$s = \gamma v \lambda_o \left(t-\frac{x}{v}\right) e^{-a\left(t-\frac{v}{x}\right)}$$

with two constants to be determined by experiment.

Taking his own observations and assuming $\gamma = 150$ pounds per cu. ft., $v = 1.16$ feet per hour, and $x = 3.38$ feet, the values $\lambda_o = 0.93$, and $a = 0.12$, are obtained. As limits (corresponding to before and after vibration) he obtains 0.93 and 0.36 for λ_o, and 0.10 for a in both cases. By means of graphs for different locations of x it is shown that all curves are the same except for a time offset or lag, s maximum being given when

$$x = v\left(t-\frac{1}{a}\right)$$

being independent of both x and t. This maximum is first reached when $x = 0$ at time $t = 1/a$.

A method for determining the lateral pressure where the concrete placement is intermittent and not uniform is indicated.

ACI NEWS LETTER

Vol. 16 No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1944

REPORT OF 1944 NOMINATING COMMITTEE

The 1944 ACI Nominating Committee, Frank E. Richart, chairman, Robert F. Blanks, Raymond E. Davis, A. T. Goldbeck, Ben Moreell, Charles S. Scholer, Myron A. Swayze and Morton O. Withey report the following nominations for offices whose terms expire at the 1945 convention:

President

DOUGLAS E. PARSONS, Chief Masonry Construction Section, National Bureau of Standards, Washington, D. C., an ACI Member since 1926, a Wason Research Medalist, member Program Committee 1939, Chairman 1940-41; Chairman Publications Committee since 1941 when it absorbed the responsibilties of the Program Committee. He became a member of the Board of Direction in 1941 when he was appointed by the Board to fill a vacancy in the position of Director-at-Large. He was elected Vice-President in 1943 and re-elected 1944.

Vice-Presidents

HARRISON F. GONNERMAN, Manager Research Laboratory, Portland Cement Association, Chicago, an ACI Member since 1918; Wason Research Medalist, Member Board of Direction as Director Fifth District since February 1943, now serving his second term in that office. A further record of his Institute activity will be found under his name in the ACI Directory. He was recently appointed to Membership on the ACI Publications Committee.

STANTON WALKER, Engineering Director, National Sand & Gravel Association and of the National Ready Mixed Concrete Association, Washington, D. C.; an ACI Member since 1921; member of Standards Committee since 1937 and of the Advisory Committee, as Chairman Department 200, General Properties, since 1938; Member Board of Direction since 1940 when he was elected Director Fourth District by appointment to fill a vacancy and twice elected; elected Director-at-Large for a three-year term, 1943.

Regional Director, First District (to succeed himself)

HENRY L. KENNEDY, Manager Cement Section, Dewey & Almy Chemical Co., Cambridge, Mass.

Regional Director, Second District (to succeed himself)

MYRON A. SWAYZE, Director of Research, Lone Star Cement Corp., Hudson, N. Y.

Regional Director, Third District

ALEXANDER FOSTER, JR., Vice-president, Warner Co., Philadelphia; ACI Member since 1912.

Regional Director, Fourth District

FRANK H. JACKSON, Senior Engineer of Tests, Public Roads Administration, Washington, D. C.; ACI Member since 1924; a member Advisory Committee as Chairman Department 900, Joint Efforts since 1929; member Program Committee 1938-40; member Publications Committee since 1941; member Board of Direction as Director Fourth District, 1937-38.

Regional Director, Fifth District

CHARLES S. WHITNEY, Consulting Structural Engineer, Milwaukee, Wis., ACI member since 1920; Wason Medalist for Most Meritorious Paper.

Regional Director, Sixth District

HERBERT J. GILKEY, Head Department Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.; ACI Member since 1924; Wason Medalist for Most Meritorious Paper; Chairman Standards Committee since March 1944; member Publications Committee since 1939; Member Board of Direction, as Director Sixth District, 1937-38.

Director-at-Large (to succeed himself)

ROBERT F. BLANKS, Materials and Testing Engineer, Bureau of Reclamation, Denver, Colo.; ACI Member since 1932; member Publications Committee since 1941; Member Board of Direction as Director Sixth District, 1941-42; appointed by Board March 1944 to be Director-at-Large to fill a vacancy; Chairman Committee 613, which reported "Recommended Practice for the Design of Concrete Mixes," now become a Standard of the Institute.

Nominating Committee

The Nominating Committee also presented 20 candidates from among whom five are to be chosen to serve with the three latest past-presidents as the 1945 Nominating Committee—the candidate receiving the most votes on the letter ballot to be Chairman. The 20 candidates: T. C.

Powers, R. D. Bradbury, W. F. Kellermann, V. P. Jensen, P. J. Freeman, L. H. Tuthill, I. L. Tyler, H. F. Clemmer, H. S. Meissner, A. R. Lord, J. C. Pearson, M. N. Clair, B. W. Steele, T. E. Stanton, J. R. Nichols, F. H. Jackson, R. B. Young, C. A. Willson, F. V. Reagel, H. J. Gilkey.

WHO'S WHO in this JOURNAL

William D. Bigler

who writes (p. 1) about the importance of the choice and arrangement of words to convey accurate meaning, was graduated from the engineering college of the University of Santa Clara, Calif., May, 1927, BS in Civil Engineering. His first position was in a survey party of the Pacific Gas & Electric Co. of San Francisco staking out power pole and tower lines in central California and later working on the preparation of the site and quarries for the P. G. & E. Salt Springs Dam in Amador County.

As an investment project, he built bungalows in Santa Maria, California, during 1929. From the spring of 1930 to the end of 1931 he designed concrete structures, drew plans for water pipe lines, and inspected construction for the Board of Water Supply of Honolulu. He was engineer for the Water Department of the City of Santa Barbara, Calif., from 1935 to 1940, preparing plans for additions to the water system and supervising a WPA crew engaged in mapping the city's mains and waterworks structures. He spent the year 1940-41 at the University of Southern California in Los Angeles as a graduate student studying journalism and English.

From 1941 to 1943 he was assistant construction engineer at the office of the Post Engineer, Fort Rosecrans, San Diego. For the past year he has taken part in the Navy's expansion program at Roosevelt Base, Terminal Island, Calif., under the Public Works Officer, by designing buildings and machine-shop cranes as associate structural engineer. His interests outside the engineering field are English and European languages and he has spent much time in studying syntax and usage of words.

Floyd B. Hornibrook

presents his first paper to the Institute (p. 13). Born at Thorp, Wash., obtained a B.S. in chemistry at the State College of Washington in 1930, after having taken undergraduate work also at Williamette University and at the University of Oregon. He obtained an M.S. in physical chemistry from the University of Maryland in 1934 from part time studies. He has been continuously employed at the National Bureau of Standards since 1930 Projects upon which he has worked during that time include the calorimetry of cement, the development of an apparatus for measuring the vibration in concrete during compaction by vibration, and the application of this apparatus in the laboratory and in the field; the development of a sonic apparatus for measuring the dynamic modulus of elasticity of concrete, and the application of this apparatus to durability studies of concrete He has been author or co-author of reports on each of these projects As a member of the American Concrete Institute since 1940 he is a member of Committees 210 and 609 and Chairman of Committee 212, Admixtures in Concrete He is also a member of the American Society for Testing Materials and its Committee C-9, and an Associate of the Highway Research Board and a member of the Project Committee on Durability of Concrete, and Chairman of the Committee on Dynamic Tests of Concrete He is the author of two discussions appearing in the JOURNAL,

one concerning concrete failures caused by low coefficient of thermal expansion, and the other on the effect of time of haul on properties of ready mixed concrete.

Paul Rogers

whose paper appears p. 21, was graduated in 1935 with the equivalent of B S. and C. E. degrees in civil engineering; and practised structural engineering in Nagyvarad, Hungary, specializing in reinforced concrete from 1935 to 1939. In 1939 he came to this country and was associated with the Public Utility Engineering and Service Corporation in Chicago, Illinois, designing power plants. Subsequently he was in charge of the structural department of the A. J. Boynton & Co., Chicago, working on the Kaiser Shipbuilders' and Columbia Steel Company's new steel plants from 1942 to 1944. From 1940 to 1944 he was also assistant Professor of Structural Engineering at the Chicago Technical College. Since May this year he has been employed as structural engineer with Hiram Walker & Sons, Inc., distillery, Peoria, Ill.

Bean and Tregoning

collaborate in a paper (p. 37), each a first-time ACI contributor.

Leonard Bean, born March 30, 1906, in Canton, China, received his B.A. from Indiana Central College in Indianapolis. From 1935 to 1941 he was employed as a chemist by Sherwin-Williams Co. at Chicago, working in the field of azo dyes and dye intermediates. During this period he took part-time graduate work at the University of Chicago. For the past three years he has served as a chemist in the Cement Section of the Clay and Silicate Products Division of the National Bureau of Standards at Washington, D. C. Work at the Bureau has included studies on air-entraining agents, solubility of portland cement in water, and the action of alkalies upon aggregates.

J. J. Tregoning went to the National Bureau of Standards in September, 1935, and analyzed cements for the Tygart river project for two years. Then followed work on the analysis and improvement of calking compounds until 1939. The next three years were spent assisting work already in progress on the liquid phase of cement pastes and the determination of alkalies. in cements. Work was begun on the action of alkalies upon different types of aggregates just before being transferred to war work.

Three New Standards: for Cast Stone, Concrete Mix Design and Pavements and Bases

The 1944 Convention voted to adopt, subject to ratification by letter ballot three new Standards as proposed by Committees 704, for Cast Stone (ACI JOURNAL, Feb. 1942), 613. for the Design of Concrete Mixtures (ACI JOURNAL, Nov. 1943—as revised in News Letter p. 3 and 38, Apr. 1944); and 617, for Concrete Pavements and Bases (ACI JOURNAL, Nov. 1943 as editorially revised on the letter ballot). On the letter ballot all three propositions prevailed by a wide margin, a two-thirds affirmative vote, among those voting, being required.

Benjamin Wilk and L. G. Lenhardt, tellers appointed by President Crum, reported the canvass of ballots to show:

1—Proposed "Specifications for Cast Stone"—yes 452, no 9.

2—Proposed "Recommended Practice for the Design of Concrete Mixes" yes 467, no 25.

3—Proposed "Specifications for Concrete Pavements and Bases" yes 458, no 19.

41st Annual

ACI Convention

Hotel New Yorker

New York

Feb. 13-16, 1945

New Members

The Board of Direction approved 107 applications for Membership in May, June and July (73 Individuals and 5 Corporations; 2 Contributing (*), 9 Junior and 18 Student) as follows:

Acosta, John Franklin, 4615 Constance Drive, San Diego 5, Calif.

Aksel, Ali, 605 E. Daniel St., Champaign, Ill.

Allabach, Earl P., Box 133, Trevose, Pa.

Ascherman, John Calvin, 409 E. Daniel St., Champaign, Ill.

Atlas, Kasim, 605 E. Daniel St., Champaign, Ill.

Austin, Chester R., Battelle Memorial Institute, 505 King Ave., Columbus 1, Ohio

Baird, J. B., 1206½ Preston Ave., P. O. Box 175, Houston 1, Texas

Barnes, S. B., 803-809 W. 3rd St., Los Angeles 13, Calif.

Barrett, J. F., 918 Harrison St., San Francisco 7, Calif.

Bastien, Paul, Dominion Square Bldg., Room 742, 1010 Ste. Catherine St. W., Montreal, Que., Canada

Bate; John F. L., Route No. 1, Box 114, Del Mar, Calif.

Bergendoff, R. N., 1012 Baltimore Ave., Kansas City 6, Mo.

Bergman, Elmer O., 1000 Fremont Ave., Alhambra, Calif.

Biberstein, Jr., Prof. Frank A., Material Testing Laboratory, The Catholic University of America, Washington 17, D. C.

*BLASCHITZ, CARLOS, Apartado de Correos No. 106, Caracas, Venezuela, South America

Brodbeck, Ernest E., P. O. Box 503, 501 Dakar Drive, Sandston, Va.

Burch, Robert, 327 Seneca Ave., Mt. Vernon, N. Y.

Burgess, R. E., 2019 6th Ave., No., Birmingham 3, Ala.

Caldwell, Charles Ira, c/o The Willows, Alpine, Calif.

Cartelli, Vincent R., 173 East 165 St., New York 56, N. Y.

Cementos Mexicanos, S. A., Apartado 392, Monterrey, N. L. Mexico (John C. Seelig)

Chapman, Allan A., 311 East Green St., Champaign, Ill.

Chinn, Allen J., 1324 V Street, Sacramento 14, Calif.

Christensen, H. C., 600 S. Michigan Ave., Chicago 5, Ill.

Clark, Arthur P., 3504 Woodley Rd., Washington 16, D. C.

*DEWEY & ALMY CHEMICAL CO., 62 Whittemore Ave., Cambridge 40, Mass.

Dolke, W. Fred, 189 W. Madison St., Chicago 2, Ill.

Dunning, Robert H., 651 Howard St., San Francisco 5, Calif.

Economy Forms Corp., Box 128, Highland Park Sta., Des Moines 13, Iowa (W. A. Jennings)

Erzen, Cevdet, 605 E. Daniel St., Champaign, Ill.

Fischer, Paul, c/o Commonwealth & Southern Corp., 600 N. 18th St., Birmingham 2, Ala.

Fisher, Frank D., 850 Mills Bldg., San Francisco 4, Calif.

Fraenkel, Dr. H. Roland, G. P. O. B. 842, Wellington Cl, New Zealand

Fromer, Milton, 404 15th St., Bradenton, Fla.

Gelbman, Louis, c/o Concrete Units, Inc., 750 Edgewater Road, Bronx, New York 59, N. Y.

Glennen, Edward J., APO 942, c/o Postmaster, Seattle, Wash.

Goodman, Charles, 313 W. 53rd St., New York 19, N. Y.

Gonzalez-G., Carlos E., P. O. Box 313, Hato Rey, Puerto Rico

Grant, J. H., c/o McCloskey & Co., P. O. Box 1728, Tampa 1, Fla.

Hanna, Fares Khalil, 7 El-Bouloc El-Farancawy, El-Charby, Old Cairo, Egypt

Hanson, Jr., E. B., 528 Second Ave., San Bruno, Calif. (Belair Shipyard)

Harris, M. H., c/o Engineering Dept., Alabama Power Co., Birmingham 2, Alabama

Helsley, Philip Warder, Route 1, Box 130, Del Mar, San Diego County, Calif.

Hetrick, L. E., Route 3, Box 264A, San Antonio, Texas

Hicks, A. W., 907 Union Planters Natl. Bank Bldg., Memphis, 3, Tenn.

Howell, Hubert G., Chief Engr., Memphis Light, Gas & Water Division, P. O. Box 388, Memphis, Tenn.

Hu, Yao-Yung, 903 W. Illinois St., Urbana, Ill.

(Ill.) Division of Waterways, Dept. of Public Works & Bldg., 201 W. Monroe St., Springfield, Ill. (Thomas B. Casey)

Jennings, Ensign Donald C., Officer Trng. School, U. S. Naval Construction Trng. Center, Sun Valley Combat Course, Davisville, R. I.

Johansen, Edwin B., 6100 Barrows Drive, Los Angeles 36, Calif.

Johnson, Harold M., 503 Marlborough Ave., Detroit 15, Mich.

Jones, William H., 6520 No. 6th St., Philadelphia 26, Pa.

Judd, Samuel, c/o Bureau of Reclamation, Denver 2, Colo.

Kale, Feyzi, 605 E. Daniel, Champaign, Ill.

Kaplan, Aaron, c/o L. Sonnenborn Sons, Inc., 88 Lexington Ave., New York 16, N. Y.

Kayayan, Bedros, 905 S. Wright St., Champaign, Ill.

Khallouf, George, Cement Works, Chekka, Lebanon, Syria

Kutchera, Ralph J., 2211-13th St., N., Wisconsin Rapids, Wis.

Lanoix, Joseph N, American Sanitary Mission, Palais des Ministeres, Port-au-Prince, Haiti

Leon, B., 407 MacLaren St., Ottawa, Ont, Canada

Levin, S. J., 1610 Park Road, N W, Room 202, Dept. 7, Washington, D. C.

Lezama y Cortina S de R. L., Gomez Farias No. 4, Mexico, D. F. (Ignacio Cortina B., Ing. Civil)

Li, Robert Ki-cheong, Dunster House F-54 Cambridge 38, Mass.

Maag, Ernst, 520 Lakeview Rd., Pasadena 2, Calif.

McFarland, Wm. H., 12 Elizabeth St., Port Dickinson, Binghamton, N. Y.

McLaughlin, Robert E., 707 27th Ave., N., St. Petersburg 4, Fla.

Miller, E. J. Jr., c/o A. O. Roder, Sunnyside, Wash.

Mintzer, Sidney, Box 458, Ancon, Canal Zone

Miyamasu, Edward, 605 E. Daniel St., Champaign, Ill.

Mullen, John J., 205 2nd Ave., Salt Lake City 3, Utah

Munilla Solino, Fernando, 13 No. 1254, Vedado, La Habana, Cuba

Munse, William H., 116 Talbot Laboratory, University of Illinois, Urbana, Ill.

Narver, D. Lee, 639 S. Spring St., Los Angeles 14, Calif.

Neve, Henry L., 6575 W. 26th Ave., Denver 5, Colo.

Ozyurt, Muhittin, 1106 W. Illinois St., Urbana, Ill.

Pfleiderer, Peter L., Jr., 176 N. Main St., Freeport, N. Y.

Phillips, A. G., Pres., The National Testing Laboratories, Ltd., 223 James St., Winnipeg, Manitoba, Canada

Prassas, Lt. M. J., Civil Engineer Corps, U. S. Navy, 1650 Harvard St., N. W., Washington 1, D. C.

Quade, Maurice N., 142 Maiden Lane, New York 7, N. Y.

Reeve, W. A., Bridge Dept., c/o John A. Roebling's Sons Co., Trenton, N. J.

Robles, Juan Ojeda, Ave. Baja California, Tijuana, Bja. Cfa., Mexico

Robinson, Russell, P. O. Box 39, Route 2, Los Altos, Calif.

Rodrigues-Derrien, A., 503 East Grew St., Champaign, Ill.

Russell and Axon, 4903 Delmar Blvd., St. Louis 8, Mo. (F. E. Wenger)

Ryan, William J., Bayshore Royal Hotel, Tampa, Fla.

Schumann, James E., 1032 Myrtle St., San Jose 11, Calif.

Shoemaker, Lewis F., Jr., 835 N. Franklin St., Pottstown, Pa.

Sorkin, Jos., c/o Howard, Needles, Tammen & Bergendoff, 1012 Baltimore Ave., Kansas City, Mo.

Spamer, Morris A., Navy Dept., Bureau of Yards & Docks, Room 2238, Washington 25, D. C.

Spofford, Charles M., 11 Beacon St., Boston 8, Mass.

Smith, Vinson B., Jr., P. O. Box 272, Gulfport, Miss.

Tanel, Fikret, 605 E. Daniel St., Champaign, Ill.

Taylor, B. G., Box 747, Rosslyn, Virginia

Taylor, L. B., 1113 Congress Bldg., Miami, 32, Fla.

Ting, Raphael E., Bridge Dept , Northern Pacific Railway Co., St. Paul 1, Minn.

Tokay, Mehmet Nejat, 105 Talbot Laboratory, Univ. of Illinois, Urbana, Ill.

True, William H., P. O. Box 241, Coral Gables 34, Fla.

Tryck, Charles W., Box 365, Anchorage, Alaska

Turkberk, Abdulgani, 807 Indiana St., Urbana, Illinois

Urul, Mehmet Vedat, 605 E. Daniel, Champaign, Ill.

Walker, W. E , Box 458, Ancon, Canal Zone

Weller, G. A., Cities Service Refining Corp., Lake Charles, La.

Wenzel, William J., Corwin & Co., Inc., Medical Arts Bldg., Great Falls, Mont.

Wilckens, Henry, 6 Russell Ave., Hazelwood Park, Adelaide, South Australia

Wycoff, Russell, 910 Commerce Bldg., Kansas City 6, Mo.

Yuce, Husnu, 605 E. Daniel St., Champaign, Ill.

Zahorik, Peter A., 936 First Wisconsin Bank, Milwaukee, Wisc.

New ACI Standards are now available in pamphlet form at 50 cents each—See p. 16.

Honor Roll

May 26 through August 21, 1944

ACI Members listed have proposed new ACI Members as shown after their names. There should be more names on this list and more new members tallied. It is all a matter of being "Member Conscious".

F. E. Richart	13
Mehmet Tokay	8
J. W. Kelly	6
Jacob J. Creskoff	5
Lewis H. Tuthill	4
Kenneth K. Knight	3
F. W. Panhorst	3
H. F. Gonnerman	2½
Hugh Barnes	2
R. F. Blanks	2
Harry Erps	2
Ernst Gruenwald	2
Fred E. Hale	2
William A. Jones	2
W. G. McFarland	2
H. W. Mundt	2
D. F. Roberts	2
Kenneth H. Talbot	2
H. B. Emerson	1½
Raymond E. Davis	1½
Bengt. Friberg	1½
Albert Haertlein	1½
Douglas E. Parsons	1½
Warren Raeder	1½
K. E. Whitman	1½
Charles E. Wuerpel	1½
Owen Arthur Aisher	1
R. Howard Annin	1
L. J. Baistow	1
J. F. Barton	1
Elmer B. Belt	1
Morgan R. Butler	1
H. F. Clemmer	1
B. M. Dornblatt	1
John J. Earley	1

Frederic Faris....1
H. M. Hadley....1
W. M. Honor....1
Norman J. Huber....1
M. E. James....1
William R. Johnson....1
Donald R. MacPherson....1
H. E. A. McCarty....1
Roscoe J. Mason....1
Ben E. Nutter....1
R. A. Plumb....1
Walter H. Price....1
Ansel T. Rogers....1
John R. Ruhling....1
M. K. Scheirer....1
Ralph A. Sherman....1
R. T. Sherrod....1
B. Skramtaiev....1
Charles Snyder....1
J. J. Waddell....1
Chas. P. Williams....1
Benjamin Wilk....1
M. O. Withey....1
E. B. Wood....1
T. Van Dyke Woodford....1

Here the credit is divided 50-50 with another Member.

J. M. Breen
Geo. C. Britton
A. E. Cooke
Theodore Crane
R. W. Crum
R. A. Crysler
G. E. Davis
R. E. Davis
Harmer Davis
R. F. Dierking
John R. Dwyer
H. H. Edwards
W. J. Emmons
H. J. Gilkey
Homer M. Hadley
P. J. Halloran
Walter N. Handy
P. W. Helsley
M. Hirschthal
L. I. Johnstone
R. R. Kaufman
John V. Konlhaas
N. M. Loney
Maurice C. Miller
W. H. Noonan
Charles O'Rourke
F. W. Paulson
Warren Raeder
H. H. Scofield
G. M. Serber
Roy T. Sessums
A. L. Strong
Sanford E. Thompson
Calvin T. Watts
Kurt F. Wendt
R. W. Winters
J. C. Witt

R. D. Bradbury and A. T. Goldbeck

have been appointed by the ACI Executive Committee to Membership on the Publications Committee for two year terms, expiring June 30, 1946. All members of the Committee, under the chairmanship of D. E. Parsons, are listed on the second white page of each JOURNAL issue.

Vice Admiral Ben Moreell

Chief of the Navy Department's Bureau of Yards and Docks received an honorary degree of Director of Engineering from the Illinois Institute of Technology last June. Admiral Moreell was president of ACI Feb. 1941 to Feb. 1942.

Frank E. Richart

Research Professor of Engineering Materials, University of Illinois and ACI Past-president, was appointed by the ACI Executive Committee to Membership on the Standards Committee.

P. H. Bates

was elected President of the American Society for Testing Materials at its June, 1944 meeting. Mr. Bates has been with the Bureau of Standards since 1910 and since 1921 Chief, Clay and Silicate Products Division. Member of ACI since 1926, he was ACI President from July, 1934, to February, 1936.

W. G. Kaiser and John A. Ruhling

The Cement Products Bureau of the Portland Cement Association, for 11 years headed by W. G. Kaiser, has been made into two bureaus: Farm Bureau, with Mr. Kaiser as manager and Housing and Cement Products Bureau with John A. Ruhling as manager. according to an announcement of William Kinney, PCA General Manager

Robert F. Blanks

received the Award of Excellence of the U. S. Dept. of the Interior "for ingenuity, efficiency and inventive talents displayed in the Government Service." Mr. Blanks is Materials and Testing Engineer, Bureau of Reclamation, Denver.

Alexander Foster, Jr.

Philadelphia, Vice President, Warner Co., has been made responsible for all operating activities of his company, except those of the Bellefonte division.

W. C. Hanna

Chief Chemist and Chemical Engineer, California Portland Cement Co., Colton, Calif. was elected in June to a two-year term to the Executive Committee of the American Society for Testing Materials.

Frederic H. Fay

of Fay, Spofford & Thorndike, Engineers, Boston, and an ACI Member for 34 years, died June 5, 1944. He was born in Marlboro, Mass. July 5, 1872. He was a brilliant student, graduating from Marlboro high school and in 1893 from Massachusetts Institute of Technology. The following year he received his master's degree from M.I.T. Long active in the Technology alumni affairs he was made president of the association in 1913 and for five years was a member of the M.I.T. corporation.

He was a member of the Boston planning board for 20 years and was the senior member of his civil engineering firm founded in 1914. He also had been vice-chairman of the Metropolitan Planning Division, member of the state planning board, and member of the New England Regional Planning Commission.

Mr. Fay was a life member of the Council of the American Institute of Consulting Engineers, of which he was president in 1927. He was a life member and former president of the Boston Society of Civil Engineers, and a director of the American Society of Civil Engineers. He was a trustee of Northeastern University and of the Dorchester Savings Bank.

Mr. Fay was a consulting engineer for many federal, state, municipal and private construction projects. Among the more important of these constructions were the Boston Army Supply Base, built during the first world war; the Bourne and Sagamore Bridges across the Cape Cod Canal, for the former of which the firm received the award of the American Institute of Steel Construction for the most beautiful bridge built at a cost of over $1,000,000 in 1934; a bridge across Lake Champlain, which received a Phebe Hobson Fowler Architectural-Engineering award in 1930; The Hampden County Memorial Bridge at Springfield, Mass.; the State Pier at Portland, Maine; the water works system of Warwick, Rhode Island; and the sewer system and sewage treatment works of Cranston, Rhode Island. During 1941-1944, the firm was the engineer member of the Architect-Engineer organization which designed 15 northern North American and off-continent bases for the U. S. War Department and during this same period was associated with other engineering firms under the title of Dry Dock Engineers, which organization designed numerous dry docks and other harbor works for the U. S. Navy Department.

John W. Kennedy

John W. Kennedy died of a sudden heart attack Saturday, July 29. Death followed within seven months his appointment as general sales manager of the Huron Portland Cement Co., Detroit, to succeed A. J. Rooney who died on Dec. 11, 1943. Mr. Kennedy had been a Member of the Institute since 1930, since 1941 as a Huron Company representative. He was two years, 1940-41, a member of the ACI Board of Direction as Director, Fifth District.

Mr. Kennedy attended the University of Michigan where he received his M.S.E. degree in June, 1922. He was on the University faculty and staff of the Michigan State Highway Department from 1921 to 1925 when he left his position as acting director of the State Highway Laboratory to become administrative assistant for the Portland Cement Association from March, 1925 to July, 1926. He then went to the Huron Portland Cement Co. and organized the technical service bureau of which he was director

John W. Kennedy

until his appointment as assistant sales manager in 1942.

Mr. Kennedy was a member, in addition to ACI, of the American Society of Civil Engineers, American Society for Testing Materials, Engineering Society of Detroit, Michigan Engineering Society, Detroit Athletic Club and the Sigma XI, Phi Lambda Epsilon and Tau Beta Pi honorary engineering and scientific societies. He served as a member of the Portland Cement Association Technical Problems Committee for the years 1938 through 1943.

He is survived by his wife, Lenore Francis, Detroit, his father, John, and a brother G. Donald, formerly Michigan State Highway Commissioner, now vice-president of the Automotive Safety Foundation.

Herman Doelman

ACI Member, 1921 to 1932 and again since 1942, died May 30 at Johns Hopkins Hospital, Baltimore, age 69. He had lived most of his life in Baltimore as an engineer in building construction, office and industrial. He was born in Holland and receiving engineering training at the Royal Institute of Engineers, Amsterdam, coming to this country in 1893, a citizen in 1898. In World War I he served as a lieutenant in the Engineer Corps. U. S. Army. He was also a member of the American Society of Civil Engineers and the American Institute of Consulting Engineers, of which he was a director.

Harry W. Mabie, Jr.

Announcement was received by the Institute in July of the death, June 5, of Harry W. Mabie, Jr., of Immel-Mabie Co., Fond du Lac, Wis. He had been an ACI Member since 1937.

Engineering Aids Wanted

The United States Civil Service Commission, Washington, D .C., announces the continuing need in the Federal service for persons to perform subprofessional engineering work in civil, topographic, photogrammetric, mechanical, and other branches of engineering. The salaries are $1,752 to $3,163 a year including overtime pay. Many appointments will be made to positions in the U. S. Geological Survey, Interior Department, the Coast and Geodetic Survey, Department of Commerce, and the Department of Agriculture. Most of the positions exist in Washington, D. C., but some may be filled in other parts of the country.

Applicants are required to have had training or experience in some branch of engineering. The grade of the position for which an applicant is considered will depend upon the amount and character of his experience or training. Detailed information regarding the requirements and duties of the positions will be supplied on request. There are no age limits for these positions and no written test will be given. Forms for applying may be secured at first- and second-class post offices, from the Commission's regional offices, or direct from the central office of the Civil Service Commission, Washington 25, D. C.

A. I. S. I. Undertakes Reinforced Concrete Research

The American Iron and Steel Institute, through its Board of Directors, recently set up a Committee on Reinforced Concrete Research, composed of the representatives of 21 producers of new billet and rail steel concrete reinforcing bars. Roy R. Zipprodt, ACI member since 1920, was selected as research and consulting engineer, to plan and supervise the investigations sponsored by the committee.

The initial program contemplates a minimum of three years with the likelihood that activities will extend over a considerably longer period. After Mr. Zipprodt's studies indicate the extent to which serious gaps exist in technical data, programs of research dealing with major problems in reinforced concrete design and construction are initiated for the specific purpose of developing new test data or of supplementing existing data on any particular phase of reinforced concrete research. The committee has made a detailed study of the various phases of reinforced concrete design and construction which, for one reason or another, should receive further investigation.

Several research projects have already been authorized. At the University of Illinois, a cooperative project is under way between the University and the Committee on Reinforced Concrete Research, covering problems related to the proper design of square and rectangular footings. The committee considers this to be a problem of major importance, no research work of any kind having been done on this phase of construction since that reported by Dr. A. N. Talbot at the University of Illinois in 1913. Major changes in basic design are anticipated as a result of these investigations.

A research fellowship at the National Bureau of Standards has been set up to study permissible unit stresses to be used in the design of reinforced concrete slabs and beams. One of the detailed problems to be investigated there will be the determination of the effect of varying ratios of slab thickness to span length on the unit stresses developed in both the concrete and reinforcing bars; also the extent to which the use of materially increased unit stresses affects deflection and consequently the extent of cracking and width of cracks. Another phase of this project will be an investigation of the effect of plastic flow in the concrete on the building up of stress between steel and concrete. In this, the effect of plastic flow on members subjected to their own dead weight only, as well as to externally applied loads, will be studied.

The committee is also engaged in the classification of a number of new or modified types of existing deformed bars which the concrete reinforcing bar industry is in the process of developing. Impetus to the industry's attempts to standardize on a single deformed bar possessed of materially improved bond characteristics was given by the W.P.B. Conservation Division's efforts to develop improved deformed bars to conserve steel during the war emergency. The standard form of bond specimen being developed by American Concrete Institute's Committee 208, Bond Stress, will be utilized to classify the various types of deformed bars submitted by producers for classification purposes. The Committee on Reinforced Concrete Research hopes to determine by a standard bond test (which the American Concrete Institute has asked its Committee 208 to report on), the extent to which the permissible design stresses in bond should be revised in existing design regulations.

Other research fellowships are to be established at well-known research laboratories and investigations will be conducted jointly by the Committee on Reinforced Concrete Research and those laboratories.

The data resulting from these investigations will then be presented to the proper code- or specification-writing bodies for approval.

Mr. Zipprodt, who has been secretary of ACI Committee 318, Standard Building Code, for a good many years, was

graduated from the University of Illinois in 1915. He has a long experience in reinforced concrete design and research beginning with his connection as assistant engineer of tests for the Joint Committee on Stresses in Railroad Track of which Dr. Talbot was chairman, and later as research engineer with the Emergency Fleet Corporation at the time extensive investigations were being made of the proper design of reinforced concrete ships. He was then at the Bureau of Standards in charge of field investigations in reinforced concrete. Subsequently he was chief structural engineer for a firm of midwestern architects and engineers, later joining the staff of Portland Cement Association as structural engineer in Chicago and then regional structural engineer attached to the New York Office of the Association. He served on the staff at Columbia University as Associate Professor of Civil Engineering and in 1940 became engaged in national defense work with the Bureau of Standards and in the office of the Quartermaster General where he had charge of standardization and simplification with particular respect to the conservation of strategic and critical materials. He joined the staff of the American Iron and Steel Institute to assume responsibility for the planning and supervision of the work of the Committee on Reinforced Concrete Research in November 1943.

ACI Construction-Practice Award

The American Concrete Institute announces the inauguration of the ACI Construction-Practice Award, to be given for a paper of outstanding merit on concrete construction practice. This award is established to honor the construction man — the man whose resourcefulness comes in between the paper conception and the solid fact of a completed structure.

The token of the award is a suitable Certificate of Award accompanied by $300 (maturity value) of United States War Bonds Series E. The object is the enrichment of the literature of concrete construction practice.

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Five cash awards are also announced for contributions to the Job Problems and Practice pages September 1944 to June 1945. For the contribution judged by an ACI Committee to rank as the best of the volume year (V. 41) — $50.00; next best $25.00; to each of the three next best $10.00.

For further particulars address Secretary American Concrete Institute, New Center Building, Detroit 2, Mich.

Special ACI Publications

Air Entrainment in Concrete

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information on ACI Membership and Publications address

AMERICAN CONCRETE INSTITUTE

New Center Building DETROIT, 2, MICHIGAN

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A reference list of advertisers who participated in the Third Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1944 issue and (when it is completed) in V. 40, ACI Proceedings.

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Admixtures for Concrete*

A REPORT BY ACI COMMITTEE 212

F. B. HORNIBROOK, Chairman

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SYNOPSIS

With the aim of providing a perspective of the field of admixtures for the use of the engineer confronted with a need of modifying concrete to meet special requirements of a given job, Committee 212 has classified admixtures into 9 broad groups. Discussions are given of the factors which might indicate the usefulness of admixtures of each group, and of the important effects which may ordinarily be expected from the use of materials of each group. The 9 groups are as follows: (1) accelerators, (2) air-entraining agents, (3) gas-forming agents, (4) natural cementing materials, (5) pozzolanic materials, (6) retarders, (7) water-repelling agents, (8) workability agents, and (9) miscellaneous.

I—GENERAL

For the purpose of this report an admixture is defined as a substance other than portland cement, aggregate or water that is used as an ingredient for concrete. According to this definition, materials are included which are added to the mixing water, which are added to the batch before or during mixing or which, if their presence in concrete significantly affects the properties of the concrete, are interground with the clinker (excepting gypsum used in the normal manufacture of cement).

Usually the purpose of using an admixture is to modify the properties of the concrete in such a way as to make it more suitable for the work in hand. Under certain conditions, the use of a suitable admixture may impart desirable characteristics which cannot be secured as economically by other methods. It is strongly emphasized, however, that no amount

*This report (received by the Institute Sept. 5, 1944), the first to be submitted by the Committee, is presented as information and for discussion only.

or kind of admixture should be considered as a substitute for the use of structurally sound well-graded concreting materials or good concreting practice.

Some of the more important modifications of properties of concrete which have been the objective of the use of admixtures are:

(a) Improvement of workability.
(b) Acceleration of the rate of strength development at early ages.
(c) Retardation of initial stiffening or increased time of set.
(d) Retardation or reduction of heat evolution.
(e) Increase in bond to steel reinforcement.
(f) Reduction in bleeding.
(g) Increase in the durability or in the resistance to special conditions of exposure to deteriorating elements.
(h) Decrease in capillary flow of water.
(i) Decrease in the permeability to liquids.
(j) In grout mixtures, the improvement of penetration and pumpability and the reduction of segregation.
(k) Prevention of settlement or creation of slight expansion in concrete and mortar used for filling blockouts or other openings in concrete structures, and in grout for seating machinery, columns, girders, etc.

In considering the role of admixtures in concrete it is pointed out that, (a) at times a change in type of cement or amount of cement used, or a modification of aggregate grading or mix proportions may offer the surest and most economical approach to the objectives desired; (b) many admixtures affect more than one property of concrete, sometimes affecting desirable properties adversely. (c) the effects of some admixtures are significantly modified by such factors as wetness and richness of mix, by aggregate grading, and by character and length of mixing. (d) The specific effects of some admixtures vary with the type and with the brand of cement used. (e) Accordingly, specific effects which will result from the use of an admixture can seldom be predicted accurately. Usually, tests of the particular lots of materials representative of those for a given job, sometimes tested under simulated job conditions, offer the only means for obtaining reliable quantitative information on the properties of concretes containing admixtures.

II—ECONOMIC ASPECTS OF THE USE OF ADMIXTURES

The use of an admixture may increase the cost of the concrete. Therefore, even though a given admixture may produce a desirable effect, the value of that effect should be weighed against its cost. Moreover, the effect of a given admixture can usually be obtained, at least in some degree, by other means or by other admixtures. Hence, whenever

possible, the cost of an admixture should be compared with that of alternative materials or methods for getting the desired result.

In evaluating an admixture, its effect on the volume of a given batch should be noted. If adding the admixture changes the volume, as is often the case, the change in the properties of the concrete will be due not only to direct effects of the admixture but also to the changes in the amounts (per cubic yard of concrete) of the original ingredients. If the admixture increases the volume of the batch, the admixture must be regarded as effecting a displacement either of part of the original mixture or of one of the original ingredients—cement, aggregate, or water. All such changes in the composition of a unit volume of concrete must be taken into account when testing the direct effect of the admixture itself, and in estimating the cost of the admixture.

Other items entering into the evaluation are the cost of handling an extra ingredient, and any effect the use of the admixture may have on the cost of transporting, placing, and finishing the concrete.

Usually the evaluation of any given material should be based on the results obtained with the particular concrete in question. This is highly desirable since the results obtained are influenced to an important degree by the characteristics of the cement and aggregate and their relative proportions.

III—CLASSIFICATION AND DISCUSSION OF EFFECTS OF ADMIXTURES

The wide scope of the admixture field, the continuous entrance of new or modified materials into this field, and the variations of specific effects with different concreting materials and conditions, precludes a detailed listing of commercial admixtures and their effects on concrete. However, a division of the admixture field into 8 broad groups, according to type of materials constituting the admixtures, or to the characteristic effects of their use, and one convenient miscellaneous group is presented along with brief statements of the general purposes and expected effects of the use of materials of each group. Listed alphabetically these groups are as follows:

1. Accelerators
2. Air-entraining agents
3. Gas-forming agents
4. Natural cementing materials
5. Pozzolanic materials
6. Retarders
7. Water-repelling agents

8. Workability agents
   (a) Finely divided minerals
   (b) Water reducing agents
   (c) Aerating materials
9. Miscellaneous
   (a) Grinding aids
   (b) Unclassified

Commercial admixtures may contain materials of two or more of the above groups. For example, a workability agent which has a retarding action may be combined with an accelerator to compensate for the retarding action. Other admixtures have properties of more than one group. A finely divided mineral, for example, may also have pozzolanic properties, but the type of the resulting effect depends largely upon the amount of material added. These types of admixtures possessing properties identifiable with more than one class are considered in the discussion following as belonging to the class that describes the most prominent or important effect on the concrete.

1. Accelerators

Accelerators may be added to concrete to increase the rate of early strength development in concrete to: (a) Speed progress by permitting earlier removal of forms; (2) Reduce the required period for curing; (3) Advance the time when a structure can be placed in service; (4) Compensate for the retarding effect of low temperatures during cold weather concreting; (5) Reduce the period of protection required for initial and final set in emergency repair and other work. In many cases the engineer must decide on the alternative procedures of using an admixture or a high early-strength cement.

Chemical materials which accelerate the normal reactions between portland cement and water include the salt, calcium chloride (the most commonly used accelerator), some organic compounds such as triethanolamine, some of the soluble carbonates, silicates and fluosilicates, and aluminous cements.

Calcium chloride generally can be used safely in amounts up to 2 and not more than 3 per cent by weight of the portland cement. Although it is sometimes added at the mixer in flake form, the preferred way is in solution in the mixing water. Laboratory tests have indicated that the increase in strength resulting from the use of 2 per cent of calcium chloride may range from 400 to 1000 psi at 1 through 7 days for 70 F curing. Limited tests at 40 F indicate that although the strengths developed through 7 days are lower, the increases in strength obtained with 2 per cent calcium chloride are of the same order as those obtained at 70 F. The increase in strength usually reaches its maximum in 1 to 3

days and thereafter gradually decreases. At 1 year some increase still continues with concretes made with most cements and it is probable that no significant decrease in ultimate strength results from the use of this admixture. The specific effect of the use of calcium chloride varies, however, for different brands of cement, as indicated by the range of strength increases cited above for the early ages. Minor effects resulting from the use of calcium chloride include a small increase in the workability of the fresh concrete, a very early commencement of the initial stiffening with some cements and accordingly, a reduction in bleeding, and a probable increase in the drying shrinkage of the hardened concrete. Data are highly conflicting on the effect of calcium chloride on the drying shrinkage, however, and it may be dependent on such factors as size of specimen and curing procedure. The rate of heat evolution is increased materially at early ages and consequently, where temperature differentials within the concrete are important factors, the effect of this property of accelerators should be taken into consideration before they are used. Also the use of accelerators in warm concrete, such as may obtain in hot weather concreting, may result in such rapid stiffening as to impede placement or finishing. Test data are available which show that calcium chloride in concrete may aggravate any tendencies toward deterioration due to adverse alkali-aggregate reaction, and may slightly reduce resistance of concrete to cycles of laboratory freezing and thawing.

General recommendations on the use of accelerators other than calcium chloride cannot be given because sufficient information on other types is inadequate. In general, other types should be used only on the basis of competent technical advice, adequate preliminary tests, or satisfactory experience with the material in question. With some accelerators, small changes in amount added cause large differences in their effect. Some are capable of reducing the period during which the concrete remains plastic to less than 10 minutes. Such acceleration lowers the ultimate strength markedly however, and is used only for emergency patching or stopping of leaks.

2. Air-entraining agents

Air-entraining agents are used principally in concrete for paving that is to be subjected to severe frost action. They cause the entrainment of very small air bubbles during the period of mixing, the action of the agent being that of a foam or froth stabilizer.

Entrained air improves the workability of fresh concrete, especially its cohesiveness, and reduces the rate and amount of bleeding. It usually permits a reduction in water content without sacrificing workability. The specific weight of the concrete containing the air-entraining

agent is less than that of the same concrete without the agent by an amount proportional to the difference in air content.

An increase in the amount of entrained air decreases compressive and flexural strength and the bond with reinforcing steel. However, by using a coarser aggregate-gradation (less sand) and by keeping the slump the same or slightly less than that required when not using an air-entraining agent, the desired strength can be maintained with little or no increase in cement content and usually with an improvement in workability. Apparently, also, such adjustments of the mix will serve to maintain the normal degree of bond with reinforcing steel. However, data on this point are meager and are not conclusive. Under average conditions the sand content can be reduced to 85 or 90 per cent of that required when not using an air-entraining agent, the decrease in sand and in the mixing water thus compensating for most or all of the volume increase attributable to entrained air.

The resistance to frost action of hardened concrete containing less than about 7 bags of portland cement per cubic yard is markedly improved by entrained air, the improvement being greater the lower the cement content. Entrained air is particularly effective in preventing surface scaling commonly caused by the use of calcium or sodium chloride for ice removal. Marked improvement is usually obtained when the entrained air is from 2 to 4 per cent of the volume of the fresh concrete.

The successful use of air-entraining agents requires some modification of usual procedures. When the agent is interground with the cement, the dry cement flows freely and sometimes is difficult to handle in conventional batching equipment and delivery trucks. After the concrete is placed in a pavement the entrained air may reduce the rate of bleeding below the rate of evaporation. When this happens, the surface soon becomes dry and sticky, making it necessary to shorten the interval between placing and finishing. Also, the number of lateral strokes per foot of forward travel of mechanical finishers can sometimes be increased to advantage. In hand finishing, steel tools are sometimes preferable to wooden ones.

The use of two air-entraining agents, Vinsol resin and Darex, is covered by The Proposed Revised A.S.T.M. Specification Designation C175—T, "Air-entraining Portland Cement for Concrete Pavements (Tentative)".*

This specification deals with air-entraining agents interground with the cement. Many other organic mixtures or compounds such as natural resins, tallows, oils, or sulfonated soaps or oils will serve as air-entraining agents in portland cement concrete. Such materials are usually effective in amounts ranging from about 0.005 to 0.05 per cent by weight of the

*Presented to the annual meeting of the A.S.T.M. in June 1944 by Committee C-1 and accepted at the meeting.

cement. Water-insoluble resins, fats, and oils are not in themselves foaming agents, but depend upon a saponification reaction with the alkali constituents of the cement for development of the foaming (or foam-stabilizing) property. These materials are effective only when inter-ground with the cement. Water-soluble agents, however, such as "neutralized" Vinsol resin (a sodium hydroxide solution of Vinsol resin) or alkali-metal salts of sulfonated oils or fatty acids are in themselves foaming agents and can be either interground with the cement or added at the mixer. Some of the latter materials form the air-entraining ingredient of proprietary mixtures available for addition at the mixer. Some air-entraining agents have little or no influence on the hydration of the cements; others do influence the hydration, usually retarding it.

The amount of air that will be entrained with a certain kind and amount of air-entraining agent can be determined only by direct test under the prevailing job conditions. Different results are obtained with different cements and aggregates and with the same materials different results are obtained under different conditions of mixing, transporting, and placing. During the progress of the work it is necessary to determine the air content frequently and to make adjustments as required to keep the air content within suitable limits. This is usually done by determining the specific weight of the fresh concrete and taking steps to hold it between 3 and 5 lb. per cubic foot less than the weight of the concrete when the air-entraining agent is absent.

3. Gas-forming agents

Settlement shrinkage or "bleeding" in plastic concrete is caused by settling of the individual solid particles in the semi-liquid mixture due to the force of gravity. The extent of settlement or bleeding is dependent on many factors and in aggravated amounts or under certain concreting conditions may result in undesirable characteristics in hardened concrete or mortar. Accumulation of low-quality matrix films, laitance layers, and voids on the underneath side of forms, blockout cavities, reinforcing steel or other embedded parts, machinery bases and at the top surfaces of concrete layers or lifts, reduce bond, water-tightness, uniformity and strength in the concrete or mortar and necessitates, in some instances, costly cleanup and grouting operations.

Aluminum powder when added to mortar or concrete reacts with the hydroxides present in hydrating cement and forms minute bubbles of hydrogen gas throughout the cement-water matrix. Usually the unpolished powder is preferred, though when a slower action is desired the polished form may be advantageous. The amounts added are usually limited to 0.005 to 0.02 percent by weight of the cement, though larger amounts may be used in the production of light-weight concrete of low strength.

The action of aluminum powder when controlled to occur during the proper interval of time, causes a slight expansion in plastic concrete or mortar and thus reduces or eliminates settlement, and may accordingly increase bond to horizontal reinforcing steel and improve the effectiveness of grout in filling joints. Ordinarily the density is slightly reduced and the resistance of the hardened concrete to frost action improved. The effect on strength depends to a considerable extent on the degree to which the tendency of the concrete to expand is restrained; without restraint the loss in strength may be considerable, but with a proper degree of restraint imposed the strength may not be affected appreciably and in some cases may be slightly increased. In hot weather the action of aluminum powder may occur too quickly and its beneficial action be lost. In cold weather the action is much slower and may not progress far enough to produce the desired effect before the concrete has set. The delayed generation of gas may or may not be harmful, depending on the rate and amount of gas generation after the concrete has set and upon the degree of restraint imposed.

4. Natural cementing materials

This group includes natural cements, hydraulic limes, water-quenched blast furnace slag and mixtures of blast-furnace slag and lime. Although blast-furnace slag itself is only weakly hydraulic, when combined with lime or other activators it becomes capable of developing significant strength.

When used, materials of this class are usually substituted for 10 to 25 per cent by weight of the portland cement, although substitution of larger amounts may sometimes be made when low strengths are not objectionable. Effects resulting from their substitution for part of the cement generally are an increase in workability, more pronounced for harsh mixes, and decreases in bleeding, in segregation, in heat of hydration and, for most of the materials, a decrease in strength. Natural cementing materials are generally considered as not having significant effect on frost resistance unless, as in the case with certain natural cements, a mild air-entraining agent is also present in the material.

The use of materials of this class may require an increase in mixing water, and accordingly, an increase in drying shrinkage may result. A relatively long curing period is needed for development of potential strength.

5. Pozzolanic materials

Pozzolanic materials are finely divided siliceous and aluminous materials which, though not cementitious in themselves, combine with hydrated lime at ordinary temperatures in the presence of water to form stable compounds of cementitious value. These materials are sometimes

used in large hydraulic structures where it is desirable to avoid high temperatures or in structures exposed to sea water or sulfate-bearing soil solutions. When used, pozzolanic materials generally are substituted for 10 to 35 per cent of the cement. They have a lower specific gravity than portland cement; therefore if substitution is made on a weight basis a greater bulk volume of cementitious material results. Substitution of pozzolanic material for part of the cement generally improves workability, reduces bleeding and segregation, reduces heat of hydration, though not in proportion to the amount substituted, improves impermeability to water and increases resistance to aggressive attack of sea water, sulfate-bearing soil solutions and natural acid waters. These effects are greater for lean mixes than for rich mixes.

The effect of substitution of pozzolanic materials for part of the cement on the strength of the concrete varies markedly with the particular pozzolanic material used. Generally the strength development is slower and is retarded more by low temperatures with the pozzolanic materials. Under favorable curing conditions, later strengths, as a rule, will be higher, though large differences result from the use of different pozzolanas. In all cases, however, prolonged wet curing is necessary for development of potential strength.

The addition of pozzolanic materials to concrete (instead of substitution for part of the cement) affects workability, impermeability, and resistance to chemical attack in a manner similar to that resulting from substitution. The improvements in workability, impermeability, and resistance to chemical attack are most marked, however, when the pozzolana is added to a concrete originally deficient in the amount of fine material; if added to one already containing an abundance of fine material, the water cement ratio must be increased, and accordingly absorption and drying shrinkage may be increased.

The effects of addition on strength vary with the mix and with the material. Generally the strength of lean mixes may be increased and the strength of rich mixes decreased.

Example of materials used in amounts ranging from 10 to 30 per cent of the weight of the cement are: fly ash, volcanic ash, heat-treated diatomaceous earths, and heat-treated or raw shales or clays.

Requirements for cements containing pozzolana are given in Federal Specification SS-C-208a, "Cement; Portland, Pozzolana."

6. Retarders

The principal uses in concrete of admixtures having a retarding effect on the set of cement are to overcome the accelerating effect of temperature during hot weather concreting operations, to remove the tendency

of some cements to exhibit premature stiffening, and to delay the early stiffening action of concrete for difficult conditions of placing. Solutions are sometimes applied to forms in order to inhibit the set of a surface layer of mortar so that it can be readily removed by brushing, thus exposing the aggregate and producing unusual surface texture effects. Retarders are especially useful in cement grout slurries, particularly where it may be necessary to redrill grout holes, for grouting over a prolonged period of time, in cases where the grout must be pumped for a considerable distance, and where hot water flows are encountered.

A wide variety of chemicals are mentioned in current literature as having a retarding influence on the normal setting time of portland cement. Others have been found variable in action, retarding the set of some cements and accelerating the set of others. Some chemicals act as retarders or inhibitors when used in certain quantities, and accelerators when used in other amounts. The effects of most of these materials on the other properties of concrete and mortar are not well known, although, in general, some reduction in strength accompanies the use of organic retarders. When used in proper concentrations, however, some retarders have been observed to have no adverse effect or to actually enhance strength.

Unless experience has been had with a retarder, its use as an admixture should not be attempted without technical advice or preferably advance experiment with the cement and other concreting materials involved, to determine the extent of its effects on the setting time and other properties of the concrete. The more commonly known retarders used as admixtures, such as carbohydrate derivatives and calcium lignosulfonate, are employed in only small fractions of a per cent by weight of the cement.

7. Water-repelling agents

Soaps or other fatty acid compounds, such as calcium, ammonium, aluminum, or sodium stearates or oleates, and petroleum oils or waxes are mixed with concrete at times to make the hardened concrete water-repellent. The water-soluble soaps are not water-repellent as added to the concrete mix, but they can become so by reaction with the hydrated lime formed during the hydration of the cement to form water-insoluble calcium soaps. The property of water-repellency may be useful in masonry mortars to decrease capillary rise of moisture, thus minimizing the discoloration and efflorescence which results at times from evaporation of capillary water. Concretes containing water-repellent materials also have application in floors or walls in contact with the soil, where the concrete is subject to capillary flow of moisture but not flow of water under a high pressure head.

Additions of water repellents in amounts usually ranging from 0.1 to 0.2 per cent, by weight of cement, increase workability of the fresh concrete, and in the hardened concrete decrease the absorption by capillarity, increase permeability under high pressure and decrease strength. The use of larger amounts increases these effects. The "waterproofed" portland cements and many of the masonry cements contain water-repellent materials.

The commercial admixtures of this class, intended to be added at the mixer, are in the form of liquids, pastes and powders containing water-repelling agents in amounts ranging from 5 to 25 per cent, the remainder being water and various fillers. Undiluted aluminum, ammonium and calcium stearates and butyl stearate emulsion are available as chemical compounds.

8. Workability agents

At times it may be desirable to increase the workability of a concrete (a) if the concrete is harsh because of aggregate grading or aggregate characteristics; (b) if the concrete must be placed around closely spaced reinforcement or in difficultly accessible sections; (c) where special means of placement are required, such as with tremie or pumping methods. It should be noted that frequently redesign of the mix or increasing the cement content may give the desired results. General effects of increasing the cement content are: increased workability, strength, impermeability, frost resistance, heat development, volume change, and increased surface crazing if the placed concrete receives much manipulation.

(a) *Finely divided minerals.* In mixes deficient in "fines" (particularly the material passing the No. 200 sieve) the addition of a finely divided mineral improves workability, reduces the rate and amount of bleeding and increases the strength. In general the higher the specific surface of the mineral the smaller the volume required to produce a given effect on workability. When an appropriate quantity of mineral powder is used, no increase in total water content of the concrete is required, and drying shrinkage and absorptivity of the hardened concrete are not much affected. The addition of a mineral powder to mixes *not* deficient in fines, particularly mixes rich in portland cement, generally decreases workability for a given water content. For that reason, the addition of mineral powders to such mixes generally entails an increase in the total water content of the concrete, and that may result in an increase in drying-shrinkage and absorptivity, and a decrease in strength.

Examples of materials added in amounts not exceeding 3 to 5 per cent by weight of the cement are bentonite clay and diatomaceous earth. Examples of materials added in larger amounts are fly ash, finely divided

silica, clay, fine sand, hydrated lime, talc, and pulverized stone, some of which may be added in amounts up to 20 per cent.

(*b*) *Water-reducing agents.* Certain organic compounds or mixtures that have been marketed under proprietary names increase the slump of concrete of a given water content and, therefore, permit concrete of a given slump to be produced at a somewhat reduced water content. Only a few such materials have been examined; hence no broad statements can be made of the general effects of all materials in this group. The materials that have been examined to date, however, contain either sulfonated organic compounds or carbohydrate salts in combination with other active or inert materials. The sulfonated type generally caused some entrainment of air and decrease in bleeding, but not to the extent as caused by materials classed as air-entraining agents. This type had a proportionately greater effect in lean mixes than in rich ones. The carbohydrate type did not cause entrainment of air and did not reduce bleeding; some retardation of set was produced.

The effect on the hardened concrete of the use of these materials, when advantage of the possible water reduction was taken, was generally somewhat improved impermeability and compressive strength, and for the type causing mild air-entrainment, an improved resistance to freezing and thawing.

As with other chemically active admixtures, the specific effect of these materials varies with different cements, and tests should be made with the job cement and preferably job materials before their use is attempted.

(*c*) *Aerating agents.* Although air-entraining agents are considered for use principally because of their effect on the durability of concrete, their effect on the workability of leaner mixes is so pronounced as to merit mention in any discussion of workability agents. The incorporation of numerous small well distributed air bubbles in the concrete acts as a lubricating medium, generally increasing the "fatness" of the mix and markedly improving the placeability of otherwise harsh concrete. As noted in the discussion of Class *2. Air-entraining agents*, entrained air improves cohesiveness and accordingly reduces segregation, reduces rate and amount of bleeding, and usually permits reduction in mixing water without loss of workability. The initial stiffening of the concrete is frequently accelerated by air entrainment. The effects of aeration on the properties of the hardened concrete are discussed under Class 2.

9. Miscellaneous

(a) *Unclassified.* There are a number of commercial admixtures which do not fit into any of the foregoing classifications. This group includes materials for which the claim is made that they increase the resistance to wear or abrasion (integral floor hardeners) or they react

with the cement or the products of hydration of the cement, filling pores, thus decreasing absorption and permeability. These materials are very difficult to evaluate, but in no case can they be considered a cure for poor workmanship.

(*b*) *Grinding aids.* Grinding aids, generally speaking, have not been considered as admixtures, and accordingly do not logically fall within the scope of this classification. By reason of their wide-spread use, however, and because of their indirect effect on concrete a brief discussion of them is included. The use of grinding aids has contributed to the finer grinding of cements. This in turn has led to higher early strengths or leaner mixes that were placeable and which yielded adequate strengths. Materials such as Vinsol resin and tallow, in amounts of the order of .03 to .06 per cent, are aids to grinding, but are generally used for the subsequent direct effect upon concrete rather than for their effect upon grinding. Water, coal and resin are the oldest used grinding aids. The commercial grinding aid TDA, when added in the amounts used as a grinding aid (limited by current A.S.T.M. and Federal specifications to .045 per cent by weight of the cement except in high-early-strength cement a maximum of .08 per cent may be used) is generally considered to have only little direct effect on concrete.

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(2) See also 4 (b)

(*b*) *Water-reducing agents*

(1) See 6 (a)

(*c*) *Aerating agents*

(1) See 2 (a), (b), and (c).

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Discussion of a report by ACI Committee 212:

Admixtures for Concrete*

By JACOB FELD, EMIL SCHMID, R. R. KAUFMAN,
DONALD MacPHERSON and F. B. HORNIBROOK, Chairman,
R. F. BLANKS and E. W. THORSON for COMMITTEE 212

By JACOB FELD†

The excellent report by Committee 212, covering as it does all possible types of admixtures brings to mind an experience in concrete production and use which may add a thought to this report.

During the construction of a large Naval Training Station, where some of the roads were of concrete and where concrete was produced in paver-mixtures, an unexpected accelerator was found to be the highly chlorinated water. The water supply system to the Station included a network of second-hand steel pipes reclaimed from previous construction projects. The only economic water source for the concrete operation was to tap these mains and to a large extent the paving operation was governed by partial completion of the water supply. Until such time as a complete section of the water distribution system was completed, the water taken from a large interior lake was found perfectly satisfactory even though it had a high salt content. However, when the pipe lines were disinfected by injection of chlorine at the pumping station until the system retained a proper residual after twenty-four hours rest, the concrete showed a rapid setting with air temperatures ranging between 50 and 60 F., that it was almost impossible to finish the surface properly. The water at that time contained about 5 parts per million of chlorine. With a chlorine content of 1 part per million, the accelerating affect was practically eliminated.

The thought arises whether chlorine content in potable water has ever been considered in tests for the setting of cement and whether chlorine solutions have the same effect on concrete as calcium chloride solutions

*ACI JOURNAL, Nov. 1944; *Proceedings* V. 41, p. 73.
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If there is no difference in action, weak chlorine solutions may be a simple and possible economical accelerator.

By EMIL SCHMID*

Progress in any field of science can be assured only if developments are investigated, facts reviewed, and the possibilities and limitations of new materials and methods established. The American Concrete Institute is to be complimented on being one of the first engineering organizations to acknowledge formally the existance of so-called "admixtures". Committee 212 has rendered a service to the construction industry by bringing the admixture question into the open and writing an informative report covering the entire field. This report forms an excellent basis for discussion and further investigation.

The claims made for admixtures, often supported by results of extensive tests, usually cover a wider field than is given under "General, A to K". Some of the effects produced by admixtures cannot be obtained by other means, regardless of economic considerations, without undesirable influences on other properties of concrete. An admixture may improve certain qualities of concrete sufficiently to make such concrete acceptable, in spite of the use of unsuitable or not well-graded aggregates or poor workmanship. This statement should not imply that admixtures can be "cure-alls", but only that in some cases their use may mean the difference between acceptance or rejection of a concrete or structure.

In addition to the modifications mentioned in the report, the following improvements, which certain admixtures can produce, might be included: 1) Increase in ultimate strength, as opposed to acceleration of the rate of strength development. 2) Increase in surface hardness, either through increased density or reduced formation of scum, or a combination of both; 3) Increase in bond to previously-placed concrete; 4) Decrease in formation of scum and laitance which may or may not be brought about by reduction in bleeding; 5) Decrease in drying shrinkage and plastic flow; 6) Improvement in the uniformity of concrete and reduction of segregation of concrete in its plastic state.

The use of a fatty, cohesive, plastic concrete does not necessarily guarantee the uniform quality of a structure. In spite of close control of the weight and quality of the ingredients and the mixing time of concrete, unavoidable variations may occur. Such changes influence the action of some types of admixtures greatly, but have little effect on the action of others.

It is advantageous to use products which decrease rather than increase the susceptibility of concrete to changes in gradation, water-cement ratio,

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mixing time, or manner of placing, regardless of whether or not they produce a fattier and more cohesive mix.

It is quite correct that the exact degree of improvement with any admixture cannot be predicted generally, and the results depend on many factors which vary from job to job. Tests can be made, or are available, which will indicate the minimum degree of improvement, if any, under anticipated conditions. These facts, however, apply to any one of the ingredients of concrete; be it aggregates or cement, and variations are not confined to admixtures alone.

It is also correct that necessary minimum results can often be obtained more economically by the use of means other than admixtures, such as by changing proportions or methods of placing. Other desirable, although not necessarily essential, benefits should be considered. In many cases, they can be obtained at a very slight increase in first cost and may be instrumental in reducing costly repair work later on. The cost of an admixture should be compared not only with the cost of the concrete itself, but with the value of the completed structure, the accessibility for repairs, the possibility and cost of shutdowns, etc.

It is stated that commercial admixtures may contain materials coming under two or more classifications. Generally speaking, the action of each constituent is independent of the action of the others. Increased ultimate strength of concrete will usually be obtained only when using a combination of a workability agent and an accelerator if the workability agent used alone will increase the strength. It will facilitate classification and evaluation if the constituents of such combinations are classified and evaluated separately, and afterwards the combined effects of the commercial product be studied. Long-terms tests are necessary to evaluate properly admixtures containing accelerators, but comparative tests of accelerating and retarding agents should only be made when the concretes in which they are used have attained a similar degree of hydration. Concrete containing an accelerator may attain 75 percent of its one-year strength (assumed end of hydration) after seven days; plain concrete, about 50 percent; and concrete containing a retarder, only 40 percent Tests at such early ages for resistance to sulphates or freezing and thawing will not be representative of the ultimate resistance of the concrete containing the retarder and will not reflect accurately what will happen in the field.

When considering strength, distinction should be made between compressive and flexural strength, as they may be influenced differently by various type of admixtures. The compressive strength usually responds more to a change in water-cement ratio than the flexural strength does, whereas; the latter may be more favorably influenced by gas-forming products.

Bleeding seems to be a very controversial subject. To a certain extent and usually within practical limits, bleeding can be controlled by changes in the proportion of coarse and fine aggregates. Entirely non-bleeding concrete is ordinarily as undesirable as one which bleeds profusely. A certain quantity of free water is necessary to finish floors, for instance. In other cases, bleeding may be of benefit by eliminating part of the surplus water needed for proper placing. Absorbent linings or other means to withdraw water after placing will not produce satisfactory results unless the concrete bleeds.

As mentioned in the report, bleeding is detrimental if it is the cause of segregation. Formation of excessive scum or laitance can be prevented by the use of certain compounds which prevent clinging of the cement and clay particles in suspension to escaping air without reducing the bleeding of clear water.

The statement that some retarders increase and others decrease strength can be explained by considering their action on the density of concrete. Retarders, which lower the surface tension of water and therefore entrain air, reduce the unit weight of concrete, particularly for concretes of unchanged water-cement ratios. Automatically, this results in a lowered strength. Other retarders, by reducing inclusion of air and possibly increasing bleeding of clear water for an unchanged water content, increase density and therefore strength.

The above also serves as an explanation to the remarks in regard to the action of the sulfonated type of admixture compared to the carbohydrate type mentioned under water-reducing agents in reference to workability and other influences. The carbohydrate type, being a non-foaming agent, will increase fluidity but not fattiness of a harsh mix and is therefore, recommended for use only in concrete containing a sufficient quantity of fines (cement or inerts) to insure proper gradation.

Air entraining agents or products decreasing the surface tension of water increase the resistance to freezing and thawing in lean mixes. The carbohydrate type will increase the resistance to freezing and thawing for lean mixes only to the extent of the reduction in water made possible by its use. Its beneficial influence on the impermeability and resistance to freezing and thawing is much more pronounced with rich concrete.

It is very difficult to classify all products in a few groups and, regardless of the way this task is undertaken, overlapping and repetition will occur. The report tries to solve this problem by classifying effects, and then mentions the class of products which have been proposed or are used to produce these effects. This classification might be supplemented by one which describes the effects which a certain class of products produces.

Such classification could be broadly subdivided into two main classes: 1) insoluble materials; 2) soluble compounds

Insoluble materials would comprise products which have no physical or chemical effect on cement during mixing, placing, or early hydration of concrete, although they may later react with certain constituents of cement, may oxidize or carbonate in the presence of moisture and air. These products are used either as pore-filling compounds to adjust the gradation of the aggregates or proposed for special purposes. Insoluble materials might be subdivided into: 1) inert compounds; 2) cementitious compounds; 3) expanding compounds.

Soluble products could be classified as: 1) products acting chemically; 2) products acting physically; 3) combinations of physical and chemical action.

Products acting chemically might be divided into: 1) accelerating compounds; 2) gas-forming compounds; 3) saponifying compounds; 4) other compounds.

An accelerator, such as calcium chloride, acts by combining chemically with the compounds contained in cement, and the main effect produced is acceleration of hardening.

Gas forming materials would be materials, which, due to a chemical action between the alkali contained in cement and themselves, form gases, which are liberated during and after mixing of concrete.

Saponification is also due to the action of alkali with the admixture. Soap is formed which entrains air during mixing only.

Chemical action is dependent on the composition of cement and the temperature and is, therefore, not uniform with all types of cement and under all conditions.

Products acting physically might be divided into products which: 1) decrease surface tension of the mixing water; 2) increase surface tension of the mixing water, or have no effect on the surface tension

Organic, soluble compounds ordinarily used to produce these effects seem to increase dispersion of cement and do not combine chemically with cement, although, by their influence on the surface and colloidal properties of cement, they may change the rate or manner of its hydration. They are added in very small percentages.

Products of a polar nature, such as sulfonated or carbo-acid compounds, influence setting time, hardening rate, density, and workability of concrete, as explained previously.

In addition to the suggestion that the present classification be supplemented by the above, it might be advisable to include in the report a section dealing with the general desirable requirements of an admixture such as; 1) It should be easy to handle and store. It should not deteriorate, harden, or become lumpy when exposed; 2) It should be distributed throughout the concrete within the regular mixing time. 3) It should be

manufactured under close control to insure uniform composition so that results are more or less predictable and only vary with job conditions. 4) An excess of the ordinarily-recommended quantity added to the concrete should not produce any marked disadvantage, and at least not harm the ultimate quality of the concrete; 5) The admixture should have a distinct color, so that, when it is added to cement, aggregates, or gaging water, unskilled labor will not add twice the quantity or none at all. 6) Results should be consistent. Variation in the degree of improvement might occur due to job conditions, but in no case and with no type of cement, should the quality of the concrete be lowered.

As a further step, it might be considered that the Committee review admixtures, if requested by manufacturers, similar to reviews now made possible by the American Society for Testing Materials for additions to cement. A summary of such reviews might be published and a file maintained by the Committee for each admixture, with the cooperation of the manufacturers.

By R. R. KAUFMAN*

In its first report the Committee has made an excellent start in the matter of publishing thoroughgoing, disinterested and constructive information, on a subject that has been left relatively untouched. Consumers will benefit directly by any program designed to established the facts in a field of growing importance, and producers of Admixtures will be helped by the decision of the Institute to assign to the subject a Committee which can become a qualified and impartial authority, lack of which in the past has resulted in reliance by some inquirers on sources of information not always qualified and impartial.

Inasmuch as this report will reach many only partly acquainted with the subject, the writer believes that some of the statements can be clarified or re-stated for the benefit of the layman, who may not understand that "variability" and "lack of desirable properties" are attributes of plain concrete mixes, otherwise there would be no need for admixtures.

The first clarification might well be of the definition of the materials included: "According to this definition, materials are included which are added to the mixing water, which are added to the batch before or during mixing or which, if their presence in concrete significantly affects the properties of the concrete, are interground with the clinker (excepting gypsum used in the normal manufacture of cement)". A superficial reading of this has left several readers with the understanding that only those admixtures which are interground in portland cement during its manufacture significantly affect the properties of the concrete. This is

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obviously not the substance of the statement, but the possibility of misinterpretation suggests the advisability of re-stating it more clearly

The fact is that, today, most of the admixtures which are interground with the cement clinker are those which affect the properties of concrete insignificantly. Examples of this type are cements interground with small amounts of fly ash, volcanic ash, oil shale or diatomaceous earth, or those which employ small amounts of grinding aids to increase the output of the grinding mills but which are of such character or amount that they do not materially affect the air entrainment characteristics of the cement. On the other hand, materials which do significantly affect the properties of the concrete, with the exception of air entraining agents, are rarely interground in the cement. As an example, calcium chloride, which has a most significant effect on concrete, is rarely interground.

The statement on page 74, "Accordingly, specific effects which result from the use of an admixture can seldom be predicted accurately. Usually, tests of the particular lot of materials representative of those for a given job, sometimes tested under similated job conditions, offer the only means for obtaining reliable quantitative information on the properties of concrete containing admixtures", is one which may lead the reader to believe that the use of admixtures increases rather than reduces the variability of concrete. Once the characteristics of a mix are known, the results from the addition thereto of the admixtures can in most cases be predicted with greater certainty than can the results produced by a normal mix consisting of the usual variable factors; [1] portland cement, sand, aggregate, mix design and labor. For example, knowing the gradation of the aggregate and mix design, the effect of the use of small amounts of finely divided materials such as fly ash, diatomaceous earth, talc or oil shale can be quite definitely predicted. The effects of triple-pressed stearic acid paste can also be predicted about as accurately as the effect of any particular brand or lot of portland cement.

Under Group 2 - Air Entraining Agents, p. 78, the statement is made, "However, by using a coarser aggregate-gradation (less sand) and by keeping the slump the same or slightly less than that required when not using an air-entraining agent, the desired strength can be maintained with little or no increase in cement content and usually with an improvement in workability". Most authorities on air entrained concrete report that mixes containing 5½ to 6½ sacks of cement per yard with the sand aggregate ratio reduced (from a proper sand aggregate ratio in the untreated mix, rather than an over sanded mix) and containing 3 to 4 percent of air will result in a concrete which will have 10 to 15 percent less compressive strength than an untreated mix of equal cement content.

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[1] Discussion by S. P. Wing, "A Study of the Cause of Non-uniformity in the Compressive Strength of Concrete Pavement Cores", ACI JOURNAL, June 1942, *Proceedings* V. 38, p. 148-1.

Concerning the consistency and advantages of cement interground today with air entraining foaming admixtures, there is a considerable difference of opinion. The success of the attempt to produce an all-purpose cement with air entraining properties is a favorite topic for argument with concrete technicians and engineers familiar with the subject. Authorities have voiced the belief that the most satisfactory method of getting controlled air entrainment in concrete mixes is to add water-soluble air entraining agents at the mixer in the proportions required by that particular mix.

It has been shown by H. F. Gonnerman in ACI JOURNAL *Supplement*, Nov. 1944, that it is possible to reduce sand sufficiently in air entrained laboratory mixes to offset the retrogression in strength normally secured. This series of tests uses sand aggregate ratios as low as 22 percent (in combination with 1-in. top size gravel) which, while giving data of interest, cannot often be employed on a job.

In classifying the admixtures the title "Gas-forming agents" was selected by Committee 212 to cover materials which prevented settlement shrinkage in plastic concrete and caused expansion. In the text, specific reference is made to the grouting of machinery. The authors correctly cover the use of aluminum powder under the category of gas-forming agents but do not point out the difficulties which may be experienced and limitations of such a material and method when employed to grout heavy industrial equipment having vibration and impact. Aluminum powder when used as a gas-forming material in grouting mixtures depends upon temperature for proper reaction as is pointed out in the article. It also depends upon the rate of hydration (hydroxal ions available) of the cement much in the same manner that water-insoluble resins, fats, and oils used as foaming agents depend upon the alkali and alkaline-earth constituents and rate of hydration of the cement for proper reaction.

Therefore, the amount of the addition of aluminum powder used with various brands of cement must be determined at the temperature, water cement ratio and under the conditions it is to be used prior to its use.

The Committee should be familiar with and in a position to provide a classification for an admixture which is not gas-forming and is of the expanding type. Such an admixture has been used extensively, especially in the industrial field for grouting bases of continuous strip rolling mills, super-hyper compressors and other equipment which transmits considerable vibration and impact to the grout and therefore requires a non-shrink, high strength, ductile grout to endure. Such a type of admixture, graded metallic aggregate with agents to promote oxidation and other constituents, has been used successfully for the past 19 years.

Under Group 5—Pozzolanic materials, the statement is made very properly that, "In all cases, however, prolonged wet curing is necessary for development of potential strength." The writer believes that a warning should be added, covering the subject of pozzolanic materials where the amount used is substantial, that prolonged wet curing is essential to assure prevention of excessive cracking due to drying shrinkage which may and frequently does take place prior to the silicious pozzolanic material entering into combination with the lime of the cement.

Reference is made to the effect of a water-reducing agent (8-b) of the sulfonated type which causes mild air entrainment and effects "an improved resistance to freezing and thawing". Having seen some of the data on this point that are available to the Committee, the writer believes the quoted phrase is highly conservative. Some of the data in question appear on pages 402-5 of the Feb. 1945, issue of the ACI JOURNAL and show that in controlled freezing and thawing tests with 13 typical brands of cement the admixture produced concrete with a durability factor of 80 after 176 average cycles, as compared with a durability factor of 20 after an average of 50 cycles for the corresponding untreated mixes.

These same data and additional findings in tests by the same authority would seem to show that use of this particular admixture involves little or no risk should circumstances make it inconvenient or impractical to make tests "with the job cement and preferably job materials before their use is attempted". Field experience with this particular admixture in some 6,000,000 cu. yd. of concrete on private and public works in the last ten years indicates plans and specifications can call for its use with complete confidence that no variables other than those controlled by the routine design and physical tests will arise in the course of the work.

In the above, the writer has referred to a specific admixture rather widely known in order to illustrate the conservative approach the Committee has properly made to a subject new on the Institute's agenda, and for which as well as for a completeness and accuracy unusual in an initial report, it deserves the highest compliment.

BY DONALD R. MacPHERSON*

The report "Admixtures for Concrete" is indeed a worthwhile contribution to the literature; in fact it is one of the very few informative papers on the subject. Since any classification of admixtures is difficult because of the large number of different types many of which produce more than one effect in concrete, the Committee should be commended

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for its efforts in classifying admixtures in a manner which should do much to clarify this subject in the minds of all interested in the field of concrete.

It is well known that for many years past those who have opposed the use of admixtures have outnumbered those who have advocated their use. This writer concurs in the following statement that "There never have been any logical reasons for this peculiar adverse attitude."* However, this attitude has discouraged consideration of additions whose use might have brought more general improvements in concrete at an earlier date. In recent years there has been a gradual trend toward a more "open-minded" attitude relative to their use because now it has been proved, even to some of the "old-school" engineers and cement producers, that admixtures can produce outstanding effects in concrete which, heretofore, could not be obtained. Therefore, if a thorough knowledge of all aspects of concrete and its ingredients is desirable, the worth or merits of various admixtures should be a part of that knowledge. To this end the Committee's report should aid greatly.

Certain statements made in the report may need some clarification and in some instances are somewhat misleading.

The first paragraph of the report sets forth a definition of admixtures and states that "According to this definition, materials are included that are added to the mixing water, which are added to the batch before or during mixing or which, if their presence in concrete significantly affects the properties of concrete, are interground with the clinker (excepting gypsum used in the normal manufacture of cement)." That part of the definition " . . . . if their presence in concrete significantly affects the properties of concrete . . . . ", does not allow the inclusion of certain interground materials which in themselves may not signi ficantly affect the properties of concrete but under some circumstances when used with other separate additives the combination does markedly affect the properties. The writer knows of instances where such combinations resulted in concrete possessing characteristics not found in concrete containing either additive alone. Therefore, as a matter of precaution, the Committee should mention or discuss the possible effects of admixtures added at the mixer with cements interground with other additives.

It is stated under (e) on page 74 that "Accordingly, specific effects which will result from the use of an admixture can seldom be predicted accurately." With respect to this statement, it is difficult for this writer to believe that any specific effects with admixtures or specific results without admixtures can be predicted accurately.

*P. H. Bates, "Portland Cement Theories (Proven and Otherwise) and Specifications", Fifteenth Edgar Marburg Lecture, Proc. 43rd Annual Meeting A.S.T.M. Vol. 40, p. 482, 1940.

The first paragraph on page 78 is somewhat misleading in view of the published field data on strength reduction in concrete because of air entrainment. This writer does not believe that in the field, strengths can be maintained with little or no increase in cement content when using a straight air-entraining agent. Published laboratory data seem to indicate that when the amount of entrained air is within the limits of 2 to 4 percent by volume that fairly large reductions in sand-total aggregate ratio can compensate for the loss of strength due to air entrainment. On the other hand, there are very few data available which show that this can be accomplished in field operations. In other words, in the field where less rigid control can be exercised and the sand-total aggregate ratio cannot be reduced greatly, loss in strength generally results where a straight air-entraining agent is used.

It would seem that the available air-entraining agents should be divided into three distinct classifications in accordance with the function of each, namely: (1) Air-entraining agents which also contain water-reducing agents and are added separately at the mixer; (2) Air-entraining agents which, besides entraining air, also act as accelerators or catalysts in concrete and are either interground with the cement or added separately at the mixer; (3) Air-entraining agents that function only to entrain air either interground with the cement or added at mixer.

Each of these three classes of materials affects the strengths of the resultant concrete but in varying degrees. The function of the first type of air-entraining agent is to reduce the water-cement ratio sufficiently at a given cement factor to more than compensate for the strength reduction due to air entrainment of about 2 to 4 percent, with the result that increases in strength can be obtained. The function of the second type of air-entraining agent is to maintain strengths by virtue of their catalytic or accelerating effects when air entrainment is about 2 to 4 percent. The effect of the third group of air-entraining agents is well known, and since these agents only entrain air, decreases in strength can be expected with increases in the amount of air entrained.

This sub-classification of air-entraining agents would give the user a better knowledge of the types of agents available so that he alone can choose the type he wishes. Intergrinding versus separate addition of an air-entraining agent is a highly controversial subject; however, it is believed that the latter method gives the best opportunity for controlled air entrainment and that eventually this will be the recommended procedure.

Referring to the first paragraph on page 79, this writer believes that the alkaline-earth constituents should also be considered as entering into the saponification reactions of the water-insoluble resins, fats and

oils. By definition, alkali constituents in this case refer only to the monovalent elements, sodium and potassium, whereas the alkaline-earth constituents would include also the divalent element, calcium.

The statements in the second paragraph on page 83 need some modification also. Commercial admixtures of the water-repellent type contain water-repelling agents in amounts ranging from 5 to 100 percent. The diluents used in the products of low water-repelling agent content, are water and various fillers as stated in the report. The 100 percent products are in dry form and contain no diluents other than very small amounts of necessary organic emulsifiers. Besides the types of undiluted chemical water-repelling agents mentioned in the report there are innumerable others, especially organic salts of fatty acids.

CLOSURE BY MEMBERS OF THE COMMITTEE

By F. B. HORNIBROOK*

The comments by the several contributors are welcome. Several of these comments concern definitions, the method of classification and the emphases employed in several instances. Naturally these factors will be affected to a large extent by the objectives assumed and by the type of reader addressed. The keynote of the committee's approach is expressed in the opening sentence of the synopsis, — "the aim of providing a perspective of the field of admixtures for the use of the engineer confronted with a need of modifying concrete to meet special requirements of a given job—." In other words the report is addressed principally to the field engineer. The definition of admixture is based on the assumption that he is concerned only with those admixed substances that create a significant effect in the concrete, i.e. that are sold as ingredients for concrete. Mr. MacPherson points out one weakness of this definition, namely, that an admixed material such as a grinding aid which in itself may not affect the concrete, may, in the presence of other additives, exert a very definite effect. Strictly speaking, an admixed material is any added substance other than the normal ingredients of concrete. That concept is employed in the following all-inclusive definition of admixtures suggested by a source outside the committee: "Admixtures shall consist of any substance or combination of substances other than portland cement, aggregate or water added to the concrete mixtures either in the field or by intergrinding with the cement at the mill (excepting gypsum used in the normal manufacture of cement)." There are good reasons for adopting such a definition rather than the functional definition used in the committee report.

The committee report, addressed as it was principally to the field engineer, also presumes that he is thoroughly familiar with the variables

*Chairman Com. 212; National Bureau of Standards, Washington, D. C.

arising in connection with cement, aggregate, mix-design and labor. The testing of cement and aggregates are standard practice. The development of mix-design by means of trial batches is nearly always recommended. The importance of competent inspection of both materials and workmanship is thoroughly recognized. The committee report calls attention to the fact that the use of admixtures may also present variables, which certainly should be treated no less exactingly than other variables arising in concreting practice.

Of the several suggestions offered for revised classifications or for a different approach to classification some are very good but others seem more appropriate for presentation to cement chemists or to admixture specialists rather than to field engineers.

Mr. Kaufman points out one class of materials not specifically covered in the report that represents a sizeable commercial use, namely graded metallic aggregates mixed with oxidation-promoting substances. The problems relating to the suitability of this type of expanding admixture for given installations, and its control in use, probably merits a separate section.

Relative to the comments on the section of the report dealing with air-entraining agents, the report states that data are meager on the effect of air-entrainment on strength when the mix is redesigned to make sure of a coarser aggregate gradation. It is unfortunate therefore that in stating that the report is misleading on this point Mr. MacPherson did not supply specific data for the benefit of us all. As to why the sand-total aggregate ratio cannot be adequately reduced in the field—once the benefit of such a reduction is acknowledged in making use of air-entrainment—is not made clear. However, the increasing use of air-entrainment and the increasing practice of redesigning the mix to make the most of the air-entrainment should soon provide a backlog of information on the points raised.

By R. F. BLANKS*

Definitions are very often the subject for much debate and they usually compromise the several individual views of committee members. It is not surprising, therefore, that a reader might be found who will take exception to the committee's definition. It was the committee's intent to regard as an admixture anything added to concrete other than cement, aggregate and water. Any of these three ingredients is subject to adulteration prior to its use. In the case of portland cement, prior additions such as grinding aids and air-entraining agents, are permitted by existing specifications without infringing on the title of portland cement. However, the committee felt that it was justified in considering

*Member Com. 212; Bureau of Reclamation, Denver.

any of these additions to portland cement as an admixture to concrete if it significantly affected its properties. Air-entraining agents are obviously in this category; a grinding aid whose influence on concrete could not be observed would not be regarded as an admixture. Mr. Kaufman admits that his difficulty with the definition arises from a superficial reading. Definitions require careful analysis and if this is not accorded them it would be better for a hasty reader to skip them. Mr. Kaufman could have helped the committee with a suggestion for restating the definition.

The writer must also take issue with Mr. Kaufman when he states that, "most of the admixtures which are interground with cement clinkers are those which affect the properties of concrete insignificantly." This is certainly not true of Darex and Vinsol Resin, the only two air-entraining additions permitted by existing cement specifications. The only other addition permitted in portland cement to date is the grinding aid TDA and the substances mentioned by Mr. Kaufman would necessarily have to be added in very small amounts to avoid detection and consequent rejection of the cement.

Several comments on the committee's report center around the statements concerning the reduction in strength accompanying the use of air-entraining agents. There is no disagreement over the fact that the introduction of air voids in the concrete, and the consequent reduction in its unit weight, reduces its strength. This will be observed more with some air-entraining agents than with others. The difference in opinion is concerned with how much of this lost strength can be regained by taking advantage of the increased workability afforded by the entrained air, thereby reducing the water content of the mix. The committee cautiously stated that the "desired strength can be maintained," implying that if equal strength was not obtained it would nevertheless be satisfactory. Some data, gathered from the Bureau of Reclamation tests on field concrete, can be offered the reader:

Five cements, without air-entraining agents were used on one job and later air-entraining cements, manufactured from the same clinkers and ground to the same finenesses, were used. In one case air-entraining agent was also added to one of the untreated cements at the mixer. The sand-total aggregate ratio was reduced to compensate for the volume of entrained air and the water in the mix was adjusted to give the same slump found for the untreated concrete. For several of the cements the strengths of treated and untreated concretes were equal and the average 28-day strength of all the treated concretes was less than 5 percent below the strength of untreated concrete.

By E. W. THORSON*

Mr. Schmid's proposal of a supplemental classification dealing with those characteristics of an admixture which involve its "foolproofness" in handling in the field, is very worthwhile. Most of the arguments against admixtures stem from inadequacies in one or more of these points.

Issue is taken with the statement in Mr. MacPherson's 5th paragraph, that ". . . it is difficult . . . to believe that . . . specific results without admixtures can be predicted accurately." By adequate attention to details in preparation of specifications and in inspection, with regard to the basic essentials necessary in the production of quality concrete (viz, water cement ratio, aggregate gradation and quality, cement factor, mixing time, placing, and curing) specific results *can* be predicted with a degree of accuracy commensurate with present design practices.

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The Effect of Alkalies in Portland Cement on the Durability of Concrete*

By BAILEY TREMPER†
Member American Concrete Institute

SYNOPSIS

Field observations, now confirmed by laboratory tests, have demonstrated the reactive character of concrete aggregates derived from lavas of the volcanic cone of Mt. Rainier, Washington. Deterioration has been universal in concrete containing such aggregates and high-alkali cement but when low-alkali cements have been used structures are in excellent condition at ages up to 19 years. The rate of retrogression is dependent on the severity of climatic conditions.

Disintegration of another type is progressing in certain structures in Eastern Washington. Reactivity of the aggregates used is not exhibited in the sealed-can test but may be demonstrated by a combination of moist storage followed by cycles of freezing and thawing. The main, if not the sole, factor influencing disintegration in this test is the content of alkalies in the cement whether they are present initially or are added at the time of mixing.

These results point to the need of limiting the alkali content of cements for satisfactory use with many aggregates in Washington.

INTRODUCTION

In the discussion that follows, reference is made to the alkali content of cement as used in structures ranging from 6 to 19 years of age. Actual analytical data as to alkali content are not available for the cements used at that time. There are, however, good means of estimating fairly accurately the probable ranges in alkalies in the cements from the several mills involved. Since 1939, routine analyses have been made at frequent intervals and these show that the percentages of the alkalies, Na_2O and K_2O, are fairly constant in each brand and type of cement. In the absence of changes in sources of raw materials or in manufacturing processes, estimates of former alkali contents when based on analyses of recent production are more than mere guesses. References will be

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made upon this basis to high-alkali cement as that containing more than 0.90 per cent alkalies; medium-alkali cement, from 0.60 per cent to 0.90 per cent alkalies and low-alkali cement, less than 0.60 per cent alkalies.

MT. RAINIER STRUCTURES

Field observations:

In 1941[1]* the writer described the condition of concrete structures containing reactive aggregates from the Cowlitz River in the vicinity of Mt. Rainier, Washington. It was stated that those structures in which low-alkali cements were used were in excellent condition. In view of the fact that some writers[2],[3] have questioned the permanent integrity of concrete containing reactive aggregates even though the cement was low in alkalies, it is of value to note that these structures, some of them now 19 years of age, remain in nearly perfect condition.

Since 1941 the structures containing high-alkali cements have continued to deteriorate and it is evident that the progress of retrogression has not yet stopped. This is indicated by comparison of photographs and by actual measurements of cracks over periods of several years. In the more protected parts of the substructures, new cracks have appeared recently. Newer structures now appear similar to the older ones at comparable ages.

The sand and gravel in bars of the Cowlitz River is derived in part from lavas of the volcanic cone of Mt. Rainier. These lavas, largely andesitic in composition, have been suspected as constituting the reactive portion of Cowlitz River aggregate. Recent evidence tends to confirm this theory.

Aggregates consisting of sand and gravel taken from the White River were used in the construction of Deadwood Creek Bridge, a concrete arch completed in 1938. The bar from which the aggregates were taken is close to the source of the river in glaciers on Mt. Rainier and is practically undiluted with rocks from other sources.

A high-alkali cement was used in Deadwood Creek Bridge. The concrete showed signs of abnormal cracking as early as 1940 but it was not until 1942 that it became evident that the rate of deterioration was greater and that the probable condition of the concrete would be worse at equal ages than that in the structures constructed with Cowlitz River aggregates.

Laboratory studies:

At the time of writing the 1941 report[1], laboratory investigations had not progressed to the point that definite conclusions as to basic

*See bibliography at end of text.

causes were warranted. These studies now indicate rather positively that the deterioration is due to unfavorable reactions between high-alkali cements and aggregates containing Mr. Rainier lavas.

4-inch cores cut with a diamond drill from the substructures of Clear Fork Creek and Summit Creek Bridges (each constructed with high-alkali cement) were stored in the laboratory fog-room at 70 F. During a period of about one month, clear, viscous beads and small, white patches appeared in considerable number on the surfaces of the cores. The chemical analysis of these exudations is given in Table 1.

TABLE 1—CHEMICAL ANALYSIS OF EXUDED MATERIAL FROM CORES

Loss on ignition	16.1
Insoluble in water (ignited basis)	9.1
Water-soluble (ignited basis)	90.9
SiO_2	85.0
R_2O_3	2.2
CaO	2.0
Na_2O	4.4
K_2O	1 9
	95.5

It will be noted that the exuded material consisted essentially of silica and alkalies. The composition is similar to that obtained from the sides of cores from Parker Dam as reported by Meissner[4].

One core, 15 in. long, was found to have a modulus of elasticity determined dynamically of 4,800,000. This value indicates some retrogression in strength since laboratory-made specimens of similar mixtures have reached values of about 6,000,000 at the age of one year.

1x1x10-in. bars of 1:2 mortar containing sand or crushed gravel from the Cowlitz and White rivers have been stored in sealed containers, in the presence of moisture, for periods of from 24 to 40 months. Fig. 1 to 6 inclusive show the amount of expansion in these specimens. Fig. 1 gives the results to 40 months of the series previously reported[1] up to the age of 6 months.

Properties of the cements used in these tests (Fig. 2 to 6 inclusive), as well as those to be described later, are given in Table 2. For convenience in studying the data, the numbers assigned to the cements are the same as the content of equivalent alkalies expressed in hundredths of one per cent. The expression "equivalent alkalies" is used to denote the quantity "sodium oxide, Na_2O, plus 0.658 times potassium oxide, K_2O". The factor 0.658 is the molecular equivalent of K_2O in terms of Na_2O.

It will be noted in Fig. 1 to 6 inclusive that Cowlitz River aggregate when mixed with high-alkali cements expands continuously, with no

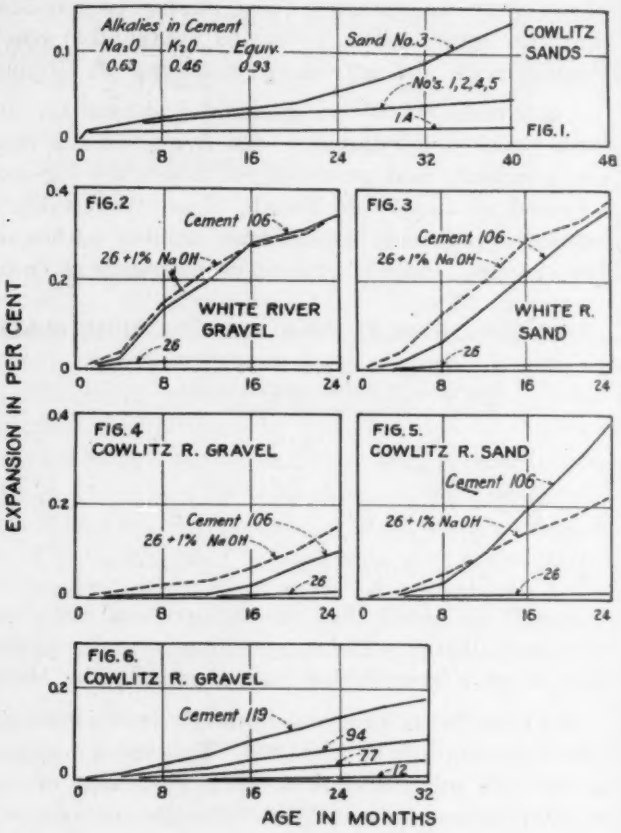

Fig. 1 to 6—Expansion of 1x1x10-in. bars of 1:2 mortar in sealed containers at 70 F.

indication of diminishing rate up to 40 months, the age of the oldest specimens. In general, aggregate from White River gives more rapid and greater expansion. It is worthy of note that in no case have specimens containing low-alkali cements expanded a significant amount. The addition of 1 per cent sodium hydroxide (*NaOH*) to low-alkali cement 26 has resulted in expansions comparable to those obtained with high-alkali cement 106.

Inspection of the group of structures containing Cowlitz River aggregate and high-alkali cement has indicated that the severity of exposure to climatic conditions has been a major influence in the rate, though possibly not the final degree, of deterioration of the concrete. Laboratory tests have been made to study the influence of freezing and thawing on the durability of concrete containing Cowlitz and White river aggregates when mixed with cements of varying alkali content.

Test specimens were 4x4x20-in. bars cast horizontally. There were four bars of each cement-aggregate combination. After 24 hours they

TABLE 2—PROPERTIES OF CEMENTS USED IN TESTS

Cement No.	26	69	84	96	106	12	39	49	68	77	94	119
SiO_2	23.55	21.80	22.00	22.04	20.69	22.61	22.33	24.67	20.53	22.33	22.43	21.01
Al_2O_3	4.40	5.36	4.92	5.12	5.89	3.86	6.36	4.24	5.37	4.82	4.97	5.55
Fe_2O_3	3.74	4.35	4.02	4.99	3.00	1.75	2.35	1.47	2.75	4.44	3.73	3.28
CaO	63.51	63.14	61.66	62.91	63.19	65.50	65.11	65.77	63.72	62.58	62.62	63.88
MgO	2.20	2.17	3.26	1.46	1.95	1.80	1.35	0.83	2.73	2.17	1.32	1.85
SO_3	1.83	1.75	1.56	1.69	1.58	2.70	1.47	1.44	2.65	1.91	1.60	1.62
Loss on Ignition	1.12	0.89	1.80	1.35	3.12	0.85	0.75	1.10	1.66	1.72	2.11	1.66
Insoluble Residue	0.20	0.27	0.28	0.10	0.20	0.25	0.30	0.20	0.27	0.50	0.32	0.28
Na_2O	0.14	0.50	0.55	0.68	0.80	0.09	0.26	0.27	0.20	0.50	0.70	0.93
K_2O	0.18	0.29	0.44	0.43	0.40	0.05	0.20	0.33	0.73	0.41	0.37	0.39
Equivalent Alkalies	0.26	0.69	0.84	0.96	1.06	0.12	0.39	0.49	0.68	0.77	0.94	1.19
C_3S	40	43	41	42	53	58	45	46	56	41	41	54
C_2S	38	31	32	32	19	21	30	36	17	33	33	20
C_3A	5	7	7	5	10	7	13	9	10	5	7	9
C_4AF	11	13	12	15	9	5	7	5	8	14	11	10
Specific Surface	1935	1600	1760	2240	1600	2520	1685	1855	1860	1790	1815	1500
Autoclave Expansion, %	0.00	0.07	0.03	0.02	0.30	−0.02	0.05	0.07	0.12	−0.01	0.04	0.17
Vicat, Initial	3–30	2–45	2–30	2–30	2–15	1–30	3–15	3–30	3–00	2–30	1–45	2–15
" Final	7–00	6–45	6–30	6–00	7–00	2–45	3–45	4–45	4–00	3–30	2–45	3–00
Gillmore, Initial	4–30	4–45	4–45	3–45	3–45	5–00	6–00	7–00	6–00	5–30	4–30	5–15
" , Final	8–00	8–00	8–00	8–00	7–30	6–30	6–30	8–00	7–00	6–30	6–00	6–15
Tensile Strength, 3 d	290	268	240	276	280	396	277	335	362	305	255	270
7 d	386	368	336	340	361	481	367	390	418	377	325	375
28 d	488	427	428	433	432		415	450	487	477	423	468
Comp. Strength 3 d	1400	1420	1385	1405	1585	3250	1455	1610	2100	1415	1100	1230
7 d	2105	2355	2325	2325	2340	4360	2490	2440	3410	2150	1895	2425
28 d	4150	4090	4575	4415	3550		4205	4060	4765	3495	3570	3890
Per Cent Floc (Paul)	0.3	0.7	0.5	0.7	0.8	1.2	0.3	1.0	2.7	0.3	0.8	0.6
Bleeding Capacity (Powers) $\Delta H'$ at w/c 6 gals. per sack, neat paste	0.024	0.047	0.052	0.034	0.039							
Sugar Test (Merriman) phenolphthalein, ml	3.5	7.8	5.5	5.5	11.1							
clear point, ml	3.9	8.8	5.9	6.0	12.7							

TABLE 3—PROPERTIES OF AGGREGATES USED IN TESTS

	Steilacoom	Irvin	Thorp	Rock Island	Cowlitz River	White River
Bulk Specific Gravity						
sand	2.67	2.65	2.67	2.66	2.62	2.66
gravel	2.67	2.66	2.75	2.72	2.64	2.59
Absorption, %						
sand	0.8	1.4	2.2	1.2	1.6	1.0
gravel	0.8	0.8	2.0	0.8	1.0	1.2
Loss in Los Angeles Machine, %						
after 100 rev.	1.4	2.1	3.3	2.5	2.4	2.9
after 500 rev.	7.0	12.0	12.1	13.8	12.3	20.2
Loss in Sodium Sulphate Soundness Test, 5 Cycles, %						
sand	0.6	0.3	1.4	0.7	1.6	0.4
gravel	0.0	1.1	0.5	0.0	3.5	0.0

were removed from the molds and stored in a fog-room at 70 F until freezing and thawing was started. They were then mounted vertically in racks with nearly unobstructed spaces of 1¼ in. between specimens. Freezing was in air to 0 F and thawing was under sprays of water to 45 F. These temperatures were at the centers of the bars as measured by thermocouples embedded in companion specimens. A typical temperature record for a 24-hour period is shown in Fig. 7. Two cycles were completed per day without interruption, except that on each seventh day one cycle was omitted while measurements were being made.

Properties of the cements and aggregates are given in Tables 2 and 3 respectively. Data of the concrete mixes are given in Table 4. The effects of freezing and thawing were measured by changes in natural frequency measured dynamically, by changes in weight, and at the conclusion of the tests, by flexural strength of the bars as beams. In this paper only the effects upon the dynamic modulus of elasticity, E (computed from natural frequency), are given because weight changes and flexural tests disclose nothing in contradiction of the dynamic results.

Fig. 8 and 9 show the results obtained with Cowlitz and White River aggregates and five cements when freezing and thawing was started at the end of a 30-day moist curing period. 77 cycles had little effect on any of the specimens. This is taken to mean that the aggregates from both sources are inherently sound and capable of producing durable concrete. After 77 cycles, freezing and thawing was discontinued and the bars were returned to moist storage at 70 F for a period of 7 months. During this storage, values of dynamic E increased from 10 to 20 per

TABLE 4—DATA OF CONCRETE MIXES FOR FREEZING-THAWING BARS

	A	B
Maximum Size of Aggregate	1¼" square	1" round
Fineness modulus		
sand	3.15	3.17
gravel	7.30	7.00
combined	5.51	5.53
Proportions per sack of cement		
lbs. of sand	250	190
lbs. of gravel	330	300
water, gallons added to room-dry agg't.		
Steilacoom	5.70	
Irvin	5.91	
Thorp	6.16	
Rock Island	6.06	
Cowlitz River	6.57	5.25
White River	6.57	
Sacks of cement per cu. yd. of concrete	5.75	6.5
Slump, inches		
minimum	3¼	1½
maximum	5½	3

Notes: Proportions shown are for bulk specific gravity of 2.67. Weights proportioned to bulk specific gravity of each material.

Each aggregate separated on ¾ in., ⅜ in., ¼ in., No. 4, 8, 16, 30 and 50 sieves and recombined to common grading.

A. Steilacoom, Irvin, Thorp, Rock Island, Cowlitz River and White River aggregates with cements 26, 69, 84, 96 and 106. Water in excess of absorption was added to prepared batches of aggregates which were stored in fog room for 24 hours prior to mixing concrete. Two 4x4x20-inch bars were molded from each batch. Two batches were made on different days. Mixed in Lancaster Type SW mixer. Bars were cast horizontally, removed from molds on following day and stored in fog room at 70 F. until freezing and thawing tests were started.

B. Cowlitz River aggregate and cements 12, 39, 49, 68, 77, 94 and 119. Aggregates in room-dry condition when concrete mixed. Four 4x4x20-in. bars from a single batch mixed in Lancaster Type SW mixer. Molded and cured as described under A.

cent above the 30-day values. Freezing and thawing was then resumed and resulted in considerable difference in durability among the concretes made with the five cements. The order of durability with either aggregate was the same for the cements. Cement 96 with 0.96 per cent equivalent alkalies was the poorest. Cement 106 was somewhat better. With this exception the order of durability is the same as that of the alkali contents of the cements. Cement 26 was virtually unaffected by freezing and thawing. The tests of part of the specimens were discontinued at the end of 155 cycles. Cement 26 was carried through 220 cycles to establish its superiority more definitely.

It is evident that some change, presumably chemical in nature, took place within the concrete during the 7-month moist storage period and

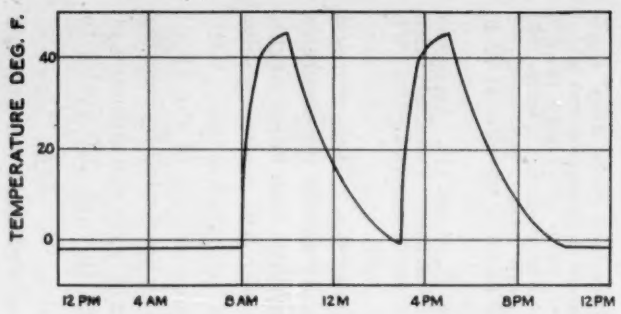

Fig. 7—Typical temperature record during freezing and thawing tests

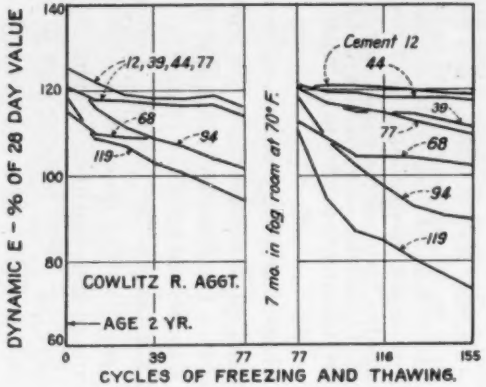

Fig. 10—Freezing and thawing tests of 4x4x20-in. concrete bars after 2 years moist storage

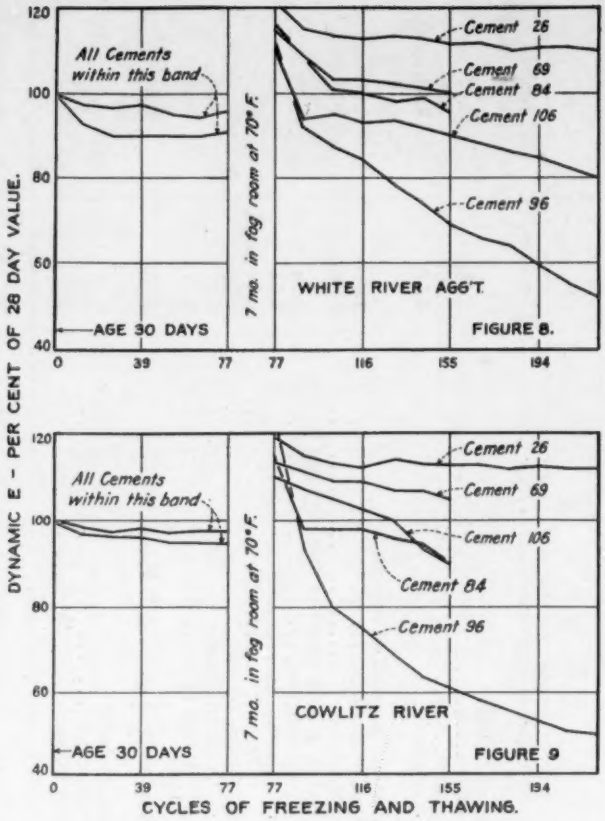

Fig. 8 and 9—Freezing and thawing tests of 4x4x20-in. concrete bars after 30 days moist storage

that the alkalies in the cements took part in this change. It should be noted that, during the original 77 cycles, the bars were subjected to intensive leaching during thawing. It is known that alkalies are leached rather readily from small concrete members. It is entirely probable, therefore, that the full quantity of the alkalies originally in the cements was not present during the 7-month storage period. Leaching could account for the greater durability exhibited by cement 106 than by cement 96 since the alkalies in some cements are more readily soluble than in others.

There were 28 bars containing Cowlitz River aggregate on hand in the laboratory which had been stored continuously in the fog room for two years. This group included 7 cements of varying alkali content (4 bars of each cement). They had been tested periodically for dynamic modulus. All of these had gained at uniform rates proportional to the logarithm of the age.

Freezing and thawing was started on these bars (at the age of two years) with the results shown in Fig. 10. During the first 77 cycles differences in alkali content of the cements were more pronounced in their effect on durability than in the previous series (Fig. 9). In general, the degree of deterioration was proportional to alkali content. As in the previous series, the bars were stored an additional 7 months in moist air at 70 F, after which freezing and thawing was resumed. The curves show that the gain in dynamic modulus which took place during the 7-month period was approximately offset by the next 13 cycles, after which the progress of deterioration continued at approximately the rate obtaining before the moist storage. This fact may be interpreted as meaning that chemical action with the alkalies in the cement had been largely completed during the initial two years of continuous moist storage at 70 F. This does not mean similar completion of chemical reactions in two years' time under field conditions of lower average temperature and periodical drying.

EASTERN WASHINGTON STRUCTURES

Field observations:

Deterioration of concrete of the same type as in the structures in the vicinity of Mt. Rainier has not been found elsewhere on the State highway system of Washington. A number of structures containing disintegrating concrete of a different type, however, are in the eastern part of the State where climatic conditions are severe. Visible defects in the latter group have characteristics which differentiate it from that at Mt. Rainier. These differences are noted in the following tabulation:

	Eastern Washington	*Mt. Rainier*
Occurrence of cracks	In superstructure only	In all parts of structure
Pattern of cracks	Longitudinal only	Longitudinal and random
Relative width of cracks	Narrow	Wide
Character of concrete adjoining cracks	Visibly disintegrated	Apparently sound although dull and lifeless when fractured
Abnormal expansion	Occasional	Frequent
Scaling on decks	General, frequently of progressive type	Rare and in thin surface layer only
Exudations	Calcium carbonate, frequently heavy	Alkali carbonate efflorescence, occasionally
D-lines	Frequent	Of rare occurrence

Three areas in Eastern Washington in which deteriorating concrete is found will be discussed in this paper. The cements used in these structures have been high or medium high in alkalies. This fact has led to investigations to study the possibility that unsatisfactory durability resulted from unfavorable reactions between high-alkali cements and the aggregates involved. There is little field evidence to support or refute such a conclusion, however. There are no cases of use of low-alkali cement with the aggregates involved although low-alkali cement when used with other aggregates in one of the areas has given good service. Nor have the results been universally poor with all structures in which the troublesome aggregates have been used. Deterioration seems to have been conditioned by severity of exposure to moisture and cycles of freezing and thawing. The fact that disintegration has occurred nearly exclusively in handrails, sidewalks, decks and wing walls illustrates the effect of exposure.

Laboratory studies:

Lacking good field evidence, dependence has been placed upon the results of laboratory studies to determine the part that alkalies in cement have had in causing deterioration. The following three sand and gravel aggregates, one from each area under discussion, were selected for laboratory study:

1. Irvin—from the Spokane River Valley about ten miles east of Spokane supplemented by fine sand from Mead. Aggregates from this general vicinity have been used extensively and have been shipped for considerable distances. Abnormal expansion in pavements containing Irvin aggregate and a cement that evidently was extremely high in alkalies has been described previously[1].

2. Rock Island—from the flood plain of the Columbia River near the town of Rock Island. Aggregates from this pit have been used in the majority of highway structures in Stevens Pass.

3. Thorp—from a bar in the Yakima River near Thorp. Aggregates from this pit were used in bridges in Blewett Pass. Although other bars in the Yakima River have yielded durable concrete, the majority of highway structures containing Thorp aggregate have been faulty.

Characteristics of the latter which may distinguish it from other Yakima River aggregate are not obvious and have not been determined.

Properties of these aggregates are given in Table 3. It will be observed that they indicate characteristics usually associated with good durability.

Mortar bar tests conducted in the same way as described for Mt. Rainier aggregates were made with the sands and crushed gravels from these pits. Negligible expansion, generally less than 0.02 per cent has resulted in every case up to ages of 2 to 4 years. This has been true with both high- and low-alkali cements and with 1 per cent sodium hydroxide added to low-alkali cement. The tests indicate that these aggregates do not react with high-alkali cements in the same manner, or at least to the same degree, as those from Mt. Rainier and they confirm the field observation that the type of deterioration is different. They might well be considered as definitely non-reactive were it not for the results of laboratory freezing and thawing tests.

Freezing and thawing tests of concrete containing these aggregates were made concurrently with, and methods were the same as, those of the Cowlitz and White River aggregates. Sand and gravel from the glacial deposit near Steilacoom on Puget Sound was included in the group. This aggregate has an excellent service record and has been used to some extent under severe climatic conditions.

The results of freezing and thawing tests are shown graphically in Fig. 11 to 14 inclusive. These data show that with one exception, to be discussed later, the first 77 cycles had very little effect on any of the specimens. This result indicates that the aggregates and cements each had characteristics making for good initial durability.

Following the first 77 cycles the specimens were stored in a fog room at 70 F for seven months. Upon resumption of freezing and thawing it became evident that changes, presumably chemical in nature, had taken place in certain specimens with the result that they were no longer resistant.

A study of the curves shows that in every case of poor resistance the cement was one of the three of highest alkali content. Good durability resulted in every case with cements 26 and 69, the lowest in alkalies. These were followed consistently in order by cements 106, 84 and 96.

The cements themselves were capable of making durable concrete as shown by the results with Steilacoom aggregate. Even cement 96 which gave such poor durability with the other aggregates was only slightly affected with Steilacoom aggregate.

Particular attention is called to the results obtained with cement 106 and Thorp aggregate. Two of the bars, No. 40 and 41, failed rapidly

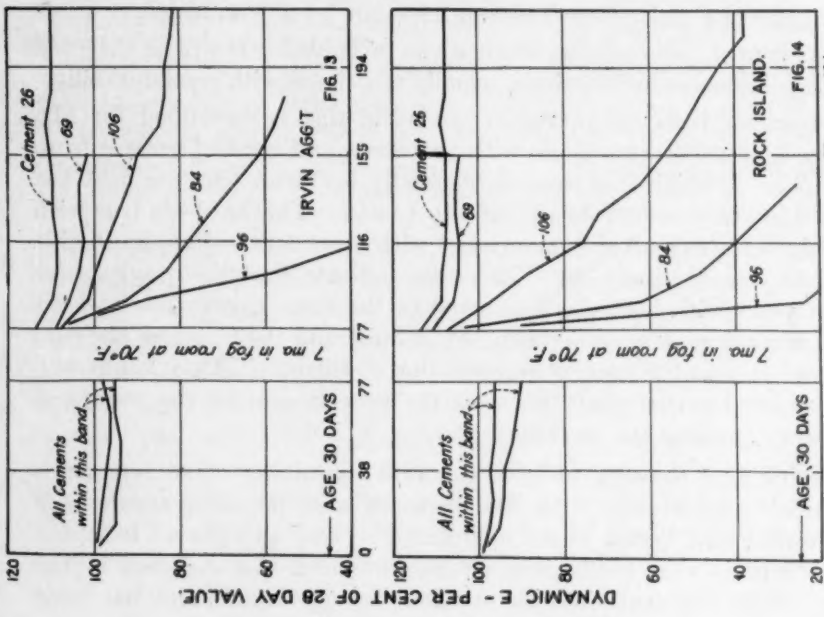

Fig. 13 and 14—Freezing and thawing tests of 4x4x20-in. concrete bars after 30 days moist storage

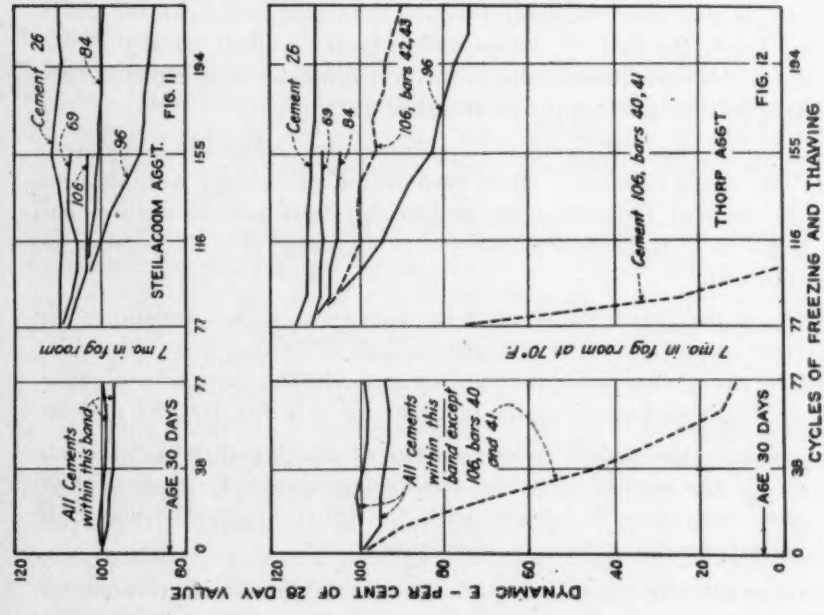

Fig. 11 and 12—Freezing and thawing tests of 4x4x20-in. concrete bars after 30 days moist storage

during the first 77 cycles. Companion bars, 42 and 43, were not appreciably affected. Bars 40 and 41 were from one batch of concrete, bars 42 and 43 from another. This is the only instance in the series of marked difference between the bars of a group. There is no apparent reason in the mix data that explains the difference in behavior of the two batches. The most plausible explanation that comes to mind, and one that has not been confirmed, is that the aggregate carries a few particles of highly reactive material that in some way were concentrated in the batch used for bars 40 and 41. These two bars presented peculiarities in appearance that have not been observed with any of the other concrete specimens. During the 7-month period of fog room storage heavy exudations, identified as calcium carbonate, appeared on the surfaces. These were similar in appearance to those observed on the Blewett Pass bridges which contained aggregate from the same source. At the conclusion of 103 cycles, when these bars were broken as beams, crystals were noted in great profusion over the entire fractured faces. Some of the crystals were in the form of hexagonal plates, others in long slender shapes. Both have been identified as calcium hydroxide. The reason for part of the calcium hydroxide crystallizing in needle form instead of the customary hexagonal plates is not known.

Fig. 13 and 14—Freezing and thawing tests of 4x4x20-in. concrete bars after 30 days moist storage

Summarizing the results of the freezing and thawing tests, it was found that

(1) Good resistance resulted with each cement when combined with Steilacoom aggregate.

(2) Good resistance resulted with the two cements of lowest alkali content when combined with each of the aggregates.

(3) Poor resistance resulted with the three cements of highest alkali content when combined with each of the three aggregates from Eastern Washington.

Fig. 11 and 12—Freezing and thawing tests of 4x4x20-in. concrete bars after 30 days moist storage

These results appear to preclude the possibility that lack of durability with the latter aggregates was influenced appreciably by factors other than high-alkali content in the cement. The tests described below also point to the alkali content of the cement as the main, if not the sole, factor determining durability.

Mention was made of 1x1x10-in. mortar bars containing sand and crushed gravel from the three eastern Washington pits which had been stored for from 2 to 4 years in sealed containers without appreciable expansion. A number of these bars were subsequently subjected to 52 cycles of freezing in air to 0 F and thawing in water to 45 F. Changes in length, weight and frequency were measured at 13 cycle intervals. The degree of deterioration indicated by the three sets of measurements

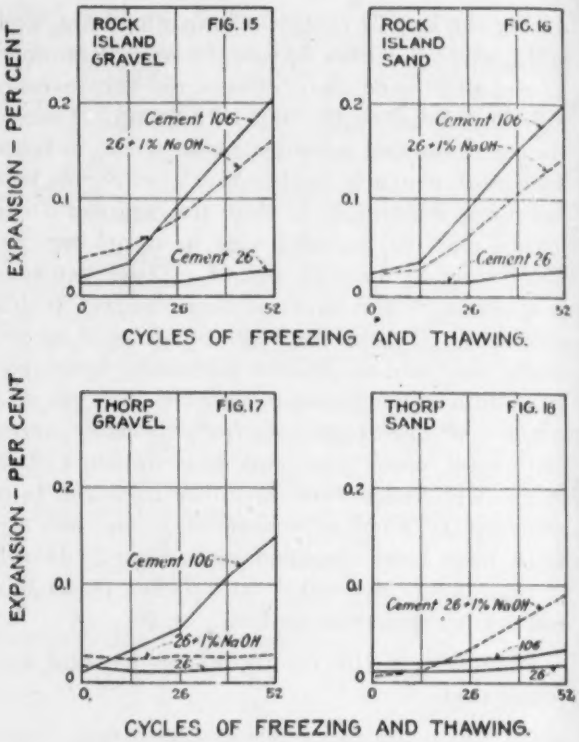

Fig. 15 to 18—Freezing and thawing tests of 1x1x 10-in. bars of 1:2 mortar after 25 months sealed storage

was in excellent agreement. Since length changes were recorded during original sealed storage these measurements only are reported for freezing and thawing. In these tests an expansion of 0.1 per cent is roughly equivalent to a loss of 20 to 25 per cent in dynamic modulus. Bars containing Irvin aggregates were not affected appreciably in these tests. This result may be due to the fact that the aggregates were crushed and graded to pass a No. 30 sieve. Sand and crushed gravel from the Thorp and Rock Island pits was graded up to the No. 4 sieve and in these specimens the effect of alkalies is well shown. These data are given in Fig. 15 to 18 inclusive. It will be noted that the expansion due to freezing and thawing was very slight when cement 26 was used with either aggregate but that cement 106 caused large expansions in three cases out of four. The addition of 1 per cent sodium hydroxide to cement 26 raised its alkali content to a value approximately equal to that of cement 106, and caused expansions of the same order as the latter cement. The inconsistencies among the specimens containing Thorp aggregate seem to be a reflection of the erratic results obtained with this aggregate in the concrete bars described earlier. Similar

instances of non-conformity to type have been noted in the structures in which this aggregate was used.

The data of the mortar bar tests bring out strongly the fact that it is the high content of alkalies and not some other property of the cement that lowers resistance to freezing and thawing. A study of Table 2 indicates a number of properties of cement 106 that might be considered to be indicative of lower quality than cement 26. It is significant, therefore, that through the addition of alkali alone the resistance of cement 26 was reduced to a degree approximating that of cement 106.

Discussion:

The aggregates from the three sources in Eastern Washington exhibit reactivity of a type different than that found at Mt. Rainier or that described by Stanton[5], Meissner[4], Kammer and Carlson[6] and others. In the laboratory, mortar bars containing these aggregates with high-alkali cement or low-alkali cement, with or without added alkali, did not expand significantly during storage at 70 F in sealed containers. Yet, among these bars, those of high-alkali content, when subjected to cycles of freezing and thawing after prolonged sealed storage, did expand appreciably.

Results similar to those of the mortar bars were obtained in freezing and thawing tests of larger concrete specimens. The cycle used in this study has been found by test to be milder than that described by Reagel[7] although conducted at similar temperatures. The difference in severity was mainly the result of thawing under sprays of water instead of by direct immersion. The latter procedure, used by Reagel and others, seems to force more water into the specimens because of the hydrostatic head involved. The cycle used in these studies was adequate in severity to disclose marked differences in durability and seems to approximate the effect of rain and melting snow during natural exposure to weather.

It is indicated, therefore, that better results would be obtained in service if cements of low-alkali content were used with aggregates similar to those of Eastern Washington that were investigated.

Reactivity of the type found in these aggregates has not been described previously. It should be noted that a period of approximately one year was the shortest time in which evidence of reactivity was established by laboratory test. It is desirable, therefore, to conduct further studies designed to develop a more accelerated test method.

CONCLUSIONS

Aggregates containing rock types derived from the lavas of Mt. Rainier react detrimentally with high-alkali cements and the type of reaction is similar to that reported by other investigators in a number

of locations. These aggregates do not react harmfully with low-alkali cements as indicated by laboratory tests and by excellent results in existing structures, some of which have reached the age of 19 years.

Aggregates from three sources in eastern Washington have produced concrete that is lacking in resistance under severe climatic exposure. Laboratory studies indicate that poor resistance is primarily a function of the alkali content of the cement. The type of reaction, however, is distinctly different from that occurring with Mt. Rainier aggregates.

These results point to the need of limiting the alkali content of cements for use with many aggregates in Washington. The present tests, however, do not furnish sufficient data to determine definitely the safe upper limit of alkalies.

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The Effect of Curing Conditions on Compressive, Tensile and Flexural Strength of Concrete Containing Haydite Aggregate*

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SYNOPSIS

Lightweight concrete has been given a severe test in the U. S. Maritime Commission's present concrete ship construction program. In its use, problems arose that could not be solved by the application of sand-gravel concrete data. This paper describes some of the strength characteristics of this type of concrete. Data herein are consistant in showing that rapid moisture loss from Haydite concrete produces a serious retrogression in the tensile and flexural strengths, regardless of the length of moist curing. This decline in strength, caused by drying shrinkage stresses developing in the outer fibers as the moisture content becomes unbalanced, is of a temporary nature and apparently can be curbed by the application of paint or membrane seal following the moist curing period. The drying shrinkage may well contribute to serious cracking in some types of structures if control is not maintained.

INTRODUCTION

This paper discusses the results of some tests of lightweight concrete made at the Belair Shipyard during the construction of twenty ship-shape concrete barges built by Barrett and Hilp under a contract with the U. S. Maritime Commission. The study was undertaken to determine the cause of extensive cracking of the concrete experienced elsewhere in similar structures.

Specific data covering the strength characteristics of Haydite concrete were meager and results obtained by different laboratories could not always be correlated. It was natural, therefore, that there should be conflicting theories regarding the cause of the cracking. The authors had designed the mix being used in the ship concrete at Belair to include 25 to 35 per cent fine Haydite and were getting highly satisfactory

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compressive strengths. There was much concern therefore upon receipt of information that laboratory results indicated that the inclusion of fine Haydite (0 to No. 4) in the mix seriously reduced the tensile strength of the concrete. The use of a minimum of 15 per cent natural sand by solid volume of the total aggregate had been specified by the Maritime Commission. This specification was included because previous tests* had shown that use of some natural sand in lightweight concrete tended to increase resistance to shattering on impact.

Cracking of the ship concrete had been negligible and for this reason the authors were reluctant to replace the fine Haydite with natural sand, and thereby reduce the cargo carrying capacity of the barges, without further proof of the necessity for doing so.

While experimenting with different proportions of natural sand in the mix, it was discovered quite by accident that the normal tensile and flexural strength of Haydite aggregate concrete is greatly dependent upon an even distribution of internal moisture at the time of testing, in fact, far more so than any strength variance influenced by the natural sand content. Data included in this paper therefore are not the results of a well planned test series based on any preconceived theory on what the results might be, but rather a series of small investigations, each individual test based on evidence newly discovered in the one just preceding it. This procedure necessitated duplication of much work in order to tie the tests together. Unfortunately the authors were not in possession of the excellent paper on this subject by W. F. Kellermann† until much of this work had been completed. Otherwise time allotted to finding the cause could have better been given to seeking a practical cure.

Few long time tests have been started because of the probability of insufficient time to complete them. Data included herein, with one exception, are based on tests requiring not more than 60 days.

TEST PROCEDURE

The Haydite used in the tests was manufactured at San Rafael, Calif., except that the fine Haydite in some batches was from Kansas City, Mo. The natural sand was a blend of 75 per cent Kaiser Top sand from Radum, Calif. with 25 per cent of a fine sand from Antioch, Calif. With few exceptions standard hull concrete, containing 50 percent coarse aggregate of size $\frac{3}{16}$ to $\frac{1}{2}$ in. and 50 per cent total fine aggregate was batched in the proportion of materials currently on hand giving a suitable grading combination. A few tests were made on mixes containing all Haydite and a few were made with no fine Haydite.

*Kluge, Ralph W. "Impact Resistance of Reinforced Concrete Slabs." ACI JOURNAL, April, 1943. *Proceedings*, V. 39, pp. 397 to 412.

†Kellermann, W. F., *Public Roads*, May, 1937.

Plastiment was added in the amount of $\frac{3}{8}$ lb. per sack of cement. The cement was type II from the Permanente and Yosemite corporations.

Specimens

Whenever practicable, a large number of specimens were cast from a routine test batch of ship concrete and divided into groups subjected to the various curing treatments. This was done to eliminate the possibility of unwanted variables between numerous small batches with a few specimens from each. Compression specimens were 6 by 12 in. cylinders, rodded and tested in accordance with standard methods. Values given are average results of from two to four tests.

Tensile specimens were cast in standard mortar briquette molds. The briquettes were cured at this laboratory and tested by a commercial testing laboratory of San Francisco. The full mix was used in the briquettes and thorough compaction was obtained by rodding with a 60 penny spike in conjunction with finger puddling. Reported results are based on averages of from two to four specimens. Briquettes breaking more than 15 per cent out of line with the average of the group were not considered in the graphs.

The procedure used in testing flexural strength was, in so far as practicable, that suggested by A. T. Goldbeck, Director, Bureau of Engineering, National Crushed Stone Association, Inc., Washington, D. C. in his paper, "Tensile and Flexural Strength of Concrete."*

The specimens were 6 x 6 x 20 in. beams, placed in two layers and compacted by rodding. The amount of rodding was varied with different consistency, but was sufficient always to assure good compaction. Loading was done as a simple beam with the load applied at third points over an 18-in. span. The finished surface was always placed in tension. The rate of loading throughout developed a maximum fiber stress of 200 psi per minute.

The temperature of specimens at the time of testing was 70 F. The testing apparatus was one of local design and construction in which consideration had to be given economy as well as the elimination of unwanted lateral forces. While it is recognized that a more elaborate device might not duplicate the actual results, it should nearly duplicate the ratio between the strength of the various groups. Moist curing was the standard 70 F. fog. Those specimens painted following the moist curing period were given 3 coats of the special concrete paint used on the ships. Three coats, applied by brush, approximated the depth of coverage the ships received.

Air curing was accomplished by storage in the laboratory in which the temperature averaged 70 F. and relative humidity 60 per cent.

*See A.S.T.M. "Report on Significance of Tests of Concrete and Concrete Aggregates".

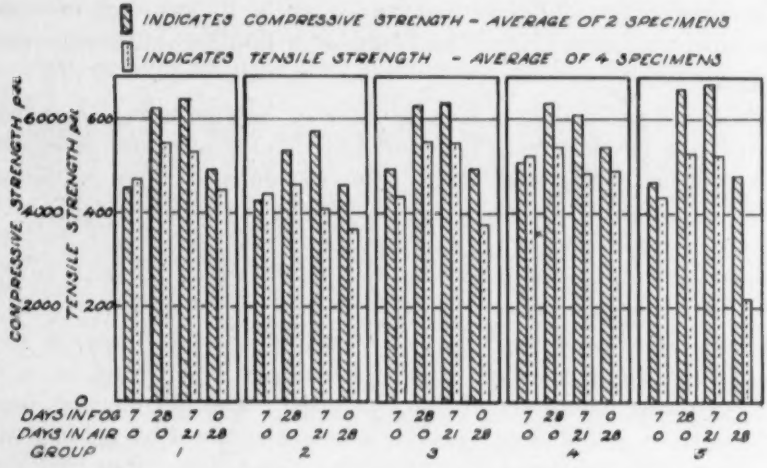

INDICATES COMPRESSIVE STRENGTH - AVERAGE OF 2 SPECIMENS
INDICATES TENSILE STRENGTH - AVERAGE OF 4 SPECIMENS
COMPRESSIVE STRENGTH psi
TENSILE STRENGTH psi
6000
4000
2000
600
400
200
0
DAYS IN FOG 7 28 7 0 7 28 7 0 7 28 7 0 7 28 7 0 7 28 7 0
DAYS IN AIR 0 0 21 28 0 0 21 28 0 0 21 28 0 0 21 28 0 0 21 28
GROUP 1 2 3 4 5

Fig. 1 (above)—Effect of drying on tensile and compressive strengths

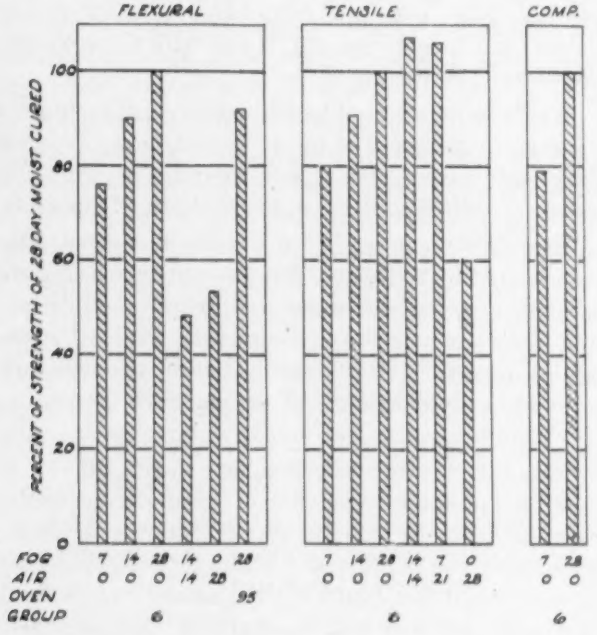


Fig. 2 (right)—Effect of drying on tensile and flexural strength of Haydite aggregate concrete after various periods of moist cure

DISCUSSION OF RESULTS

There is no evidence in any of the groups in Fig. 1 to indicate the tensile strength would be adversely effected either by drying or by variance in the amount of fine Haydite used. Likewise Fig. 2 shows no loss in tensile strength. However, Fig. 2 does show the flexural strength after 14 days of drying to be only 48 per cent of the continuously moist

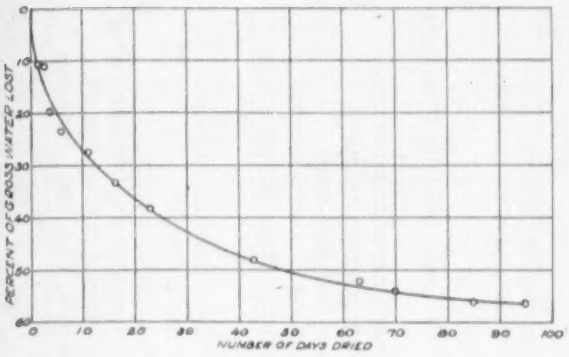


Fig. 3—Rate of moisture loss from two beams oven dried at 135 F

cured specimens. This was the first indication that an unbalanced moisture content might be at least partially responsible for the cracking in structures examined elsewhere.

Two of the beams from the group in Fig. 2, dried to constant weight at a temperature of 135 F., are shown to have recovered all but 8 per cent of their strength before drying in spite of the fact that the accelerated drying caused stress sufficient to create a myriad of minute surface cracks within two days. Undoubtedly there would have been less cracking under normal drying conditions and at constant weight the flexural strength would have been as great as, or greater than, before drying. Kellerman found specimens dried to constant weight in an electric oven at 170 F., after 360 days moist curing, to be 67 per cent as strong as they were at the end of the curing period. The condition of the surface of these beams was not noted, however.

Referring to Fig. 6, it will be seen that the moisture remaining within the small briquettes after one day of drying is approximately equivalent to that in the beams after a month of drying and that by the end of three weeks they are approaching constant weight. This explains why no recession in strength due to drying was revealed by tensile tests shown in Fig. 1 and 2. Fig. 4 shows that a decline in strength does occur in the briquettes, but because of the high ratio of surface area to volume the moisture is lost and strength regained rapidly.

It appears significant that with the short curing period of 7 days the loss in strength during drying was neither as great nor did it occur as quickly as in those specimens cured 14 days. This is shown in Fig. 4 to be true for both the beams and briquettes and is in agreement with Kellerman who found a 38 per cent retrogression in strength of beams moist cured 353 days followed by 7 days drying in conditions less acute than here. A plausible explanation is that since the modulus of elasticity increased with age, the outer fibers of the younger group, being more

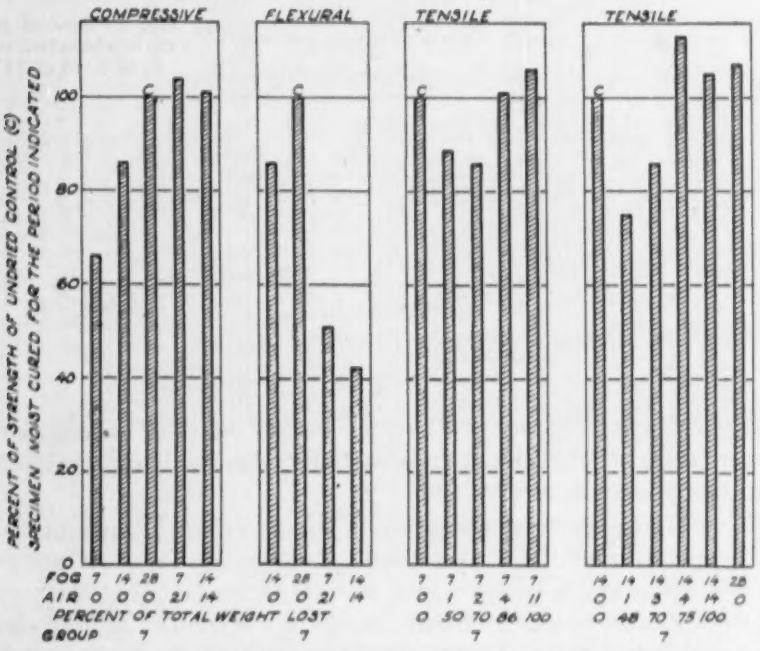

COMPRESSIVE
FLEXURAL
TENSILE
TENSILE
PERCENT OF STRENGTH OF UNDRIED CONTROL (C) SPECIMEN MOIST CURED FOR THE PERIOD INDICATED
0
20
40
60
80
100
C
FOG 7 14 28 7 14 14 28 7 14 7 7 7 7 7 14 14 14 14 14 28
AIR 0 0 0 21 14 0 0 21 14 0 1 2 4 11 0 1 3 4 14 0
PERCENT OF TOTAL WEIGHT LOST
0 50 70 86 100 0 48 70 75 100
GROUP 7 7 7 7

Fig. 4—Effect of drying on compressive, flexural and tensile strength of Haydite aggregate concrete and the relation of strength decline to moisture loss

plastic, yielded to deformation without developing as much stress as those in the group cured 14 days. Further evidence was observed in the aforementioned oven-dried beams which cracked much more after 28 days moist curing than did a test panel made at the start of the program in this yard. The test pannel was made with wetter concrete; it received no water curing; and it was exposed from the beginning to drying winds and sun.

As the tensile and flexural strength recedes during drying, the compressive strength increases. Fig. 1 and 4 show the strength of cylinders moist cured 7 and 14 days followed by 21 and 14 days respectively of drying to be consistently higher than of those continuously moist cured. This is not in strict agreement with Kellermann's findings, but the different storage conditions were possibly responsible for this variance. Although the strengths of Kellermann's dried specimens were lower than those moist cured at any given age, they continued to gain strength only slightly slower at a considerably lower temperature than his moist cured cylinders. Hydration of cement would naturally be somewhat retarded.

Evidently some initial compression loading on the partially dried cylinders serves only to bring stresses in the outer shell back into equili-

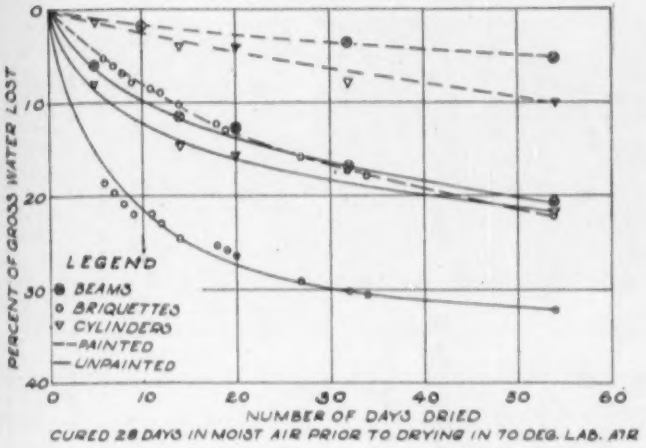
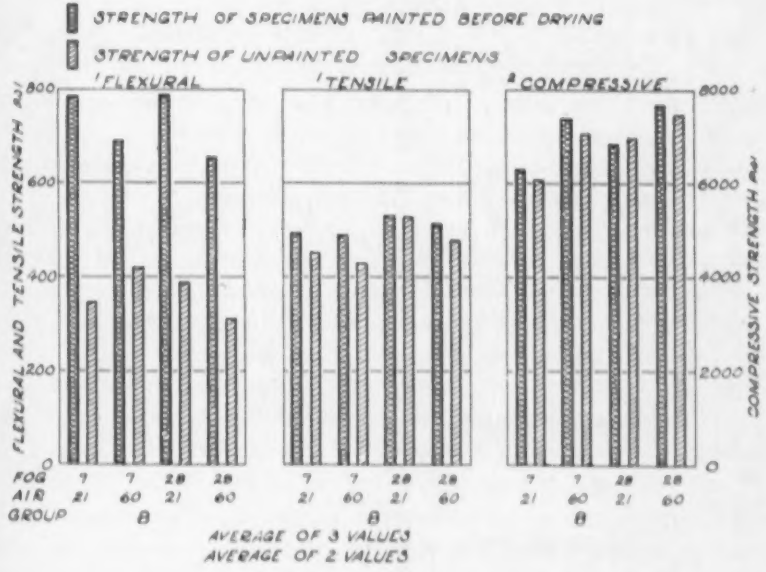


Fig. 5 (at left)—Effect of drying on the strength of specimens painted after moist curing

Fig. 6 (below)—Comparative rates of drying between specimens painted and unpainted



brium, and since at final rupture the outer portion must be first to yield, the result is a higher than normal strength. The effect on strength properties of painting the concrete immediately following the moist curing period is shown in Fig. 5 and the effectiveness of the paint in retarding the moisture loss is given in Fig. 6. Fig. 7 shows the relative appearance between the cross section of a painted and unpainted specimen after each had been dried 60 days. The photograph was taken immediately after breaking the beams. Note the even dark shading over the entire area of the painted beam in contrast with the light peri-

TABLE 1—SUMMARY OF MISCELLANEOUS DATA OF THE VARIOUS MIXES USED IN THE TEST SERIES

Test Group No.	Mix Proportions Percent of Solid Volume of Total Aggregate					Cement Content Sks. Per Cu. Yd.	Slump In Inches	Weight of Fresh Concrete Lbs. Per Cu. Ft.	Average Compressive Strength at 28 Days
	Nat. Sand	Haydite Aggregate							
		S.R ½	S.R ⅜	S.R 3/16	K.C 3/16				
1	0	30	20	50		9.17	4.0		6410
2	0	30	20		50	9.17	4.0		5780
3	15	30	20	35		9.17	4.0		6335
4	25	30	20	25		9.17	4.0		6085
5	50	30	20			9.17	4.0		6770
6	20	30	20	30		9.07	4.2	119.7	6300
7	15	30	20		35	9.20	4.0	115.0	5680
8	15	30	20		35	9.16	2.7	118.6	6130

TABLE 2—GRADING AND PHYSICAL PROPERTIES OF AGGREGATE

% Pass. Screen Size	Nat. Sand	3/16 K. C.	Haydite Aggregate		½ S. R.
			3/16 S. R.	⅜ S. R.	
½ Inch					86
⅜ Inch				99	38
No. 4			99	24	6
No. 8	87	86	72	5	
No. 16	62	46	40	1	
No. 30	43	25	23		
No. 50	20	15	14		
Pan					
F. M.	2.82	3.22	3.47	5.71	6.66
PHYSICAL PROPERTIES					
Sp. G. (Dry)	2.56	1.53	1.69	1.56	1.46
Absorption	2.4%	13%	15%	13%	13%
Wt. per Cu. Ft. (Dry-loose)	94	57	63	49	45

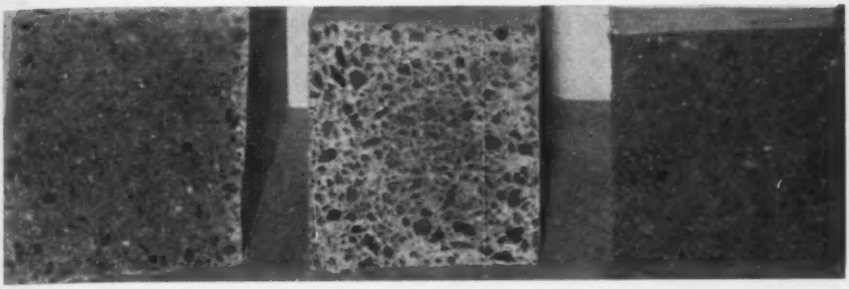


Fig. 7—Relative appearance of cross-section of painted and unpainted specimens

Specimen at left, unpainted, dried 60 days in air at 70 F.; center, ovendried to constant weight; specimen at right, painted, dried 60 days in air at 70 F.

meter of the unpainted one. It is evident from the photograph, and from Fig. 6 which shows the painted beams to have dried slowly and at a nearly even rate throughout the 60 day period, that at no time could the moisture content have been greatly out of equilibrium.

There is no proof in the data that the paint is not merely delaying the recession in strength and that it would not occur later and extend over a longer period; in fact the loss in strength noted in Fig. 5 between 21 and 60 days of drying is evidence that the paint does not entirely check the development of stress. However, the behavior of the painted briquettes, having had time to dry more than the beams within the test period, indicates that the painted beams will never drop as low in strength as the unpainted ones. Fig. 6 shows that the 28 day moist cured painted briquettes had lost approximately the same percentage of moisture after drying 21 days as had the unpainted ones after drying 3 days. If there were ever to be an appreciable decline in strength it should be in evidence at this time, yet Fig. 5 shows the strength to be normal for continuously moist cured concrete.

The importance of preventing a rapid loss of moisture from Haydite aggregate concrete, particularly in such thin walled structures as ships, cannot be too greatly emphasized. Let us assume for instance, locked shell or skin stress great enough that the shell is not quite capable of withstanding the additional working stress. Slight impetus, developed through failure of the shell might be sufficient to carry the crack completely through the section possibly with less loading than would be needed to cause failure had the shell already been cracked and stress relieved. This can be likened to the bursting of a ripe watermelon when the skin is cut.

The writers regret having to give up further search for a practical means of curbing this temporary retrogression in strength, however

another approach has suggested itself as being worthy of investigation. While experimenting with batching of the highly absorbitive Haydite in a dry instead of the normal prewetted condition we were able to reduce substantially the gross water content and thereby reduce the weight of fresh concrete by 4.0 lb. per cu. ft. After a year of moist curing there remained a 3.5 lb. differential in weight, due entirely to the difference in absorbed water. No flexural strength tests were made, but compressive strength of the dry aggregate concrete was consistently higher than that of the prewetted aggregate concrete. The water absorbed within the aggregate is strongly indicated to be responsible for the temporary loss of flexural and tensile strength. Any reduction in absorbed water that is effected by dry batching or by coating the aggregate to resist absorption might be beneficial.

CONCLUSIONS

1) It is important to prevent rapid drying of concrete made with Haydite (lightweight) aggregate regardless of the length of the moist curing period. This is particularly true for the type of structures receiving maximum loading within a few weeks after the moist curing and in which high tensile and flexural strength is a prime consideration.

2) The compressive strength of Haydite concrete is no criterion of the flexural strength except during the moist curing period. During this time the flexural strength is normally 12 per cent of compressive but it can temporarily recede to under 5 per cent during a transitional period in which drying shrinkage stress develops in the outer fibers of the concrete. This stress is subsequently relieved by continued drying to the center of the mass and the flexural strength is regained.

3) The application of paint or other membrane to the concrete following moist curing serves to maintain uniformity of moisture throughout a long drying period and is beneficial, not only in promoting continued hydration of the cement, but also in preventing development of high stresses that reduce the strength.

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A Limited Investigation of Capping Materials for Concrete Test Specimens*

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Member American Concrete Institute

SYNOPSIS

An investigation was made of the influence of type and age of cap on the apparent strength of concrete cylinders. The capping materials tested consisted of 2 types of commercial sulfur-silica compounds and 2 commercial plasters which had been in general use for this purpose. A total of 252 tests were made of concrete ranging in strengths from 2800 to 7500 psi.

The results indicate that the age of the caps has an important bearing on the apparent strength of the concrete with one hour being the minimum time required for the best of the materials to develop the full strength of the concrete.

1. INTRODUCTION

There are many factors which affect the apparent compressive strength of concrete test cylinders, and it is highly important that the testing laboratory fully develop the inherent strength of the specimens submitted to it for testing. Others have shown that concavity, convexity, irregularity and beveled-end conditions affect the apparent strength to varying degrees depending upon the capping material used.

This paper reports features of a laboratory investigation undertaken to clear up certain particular aspects of the problem of concrete-cylinder capping, and their effects on the apparent strength of the specimen tested. In some respects the work parallels features investigated by Troxell[1]‡ and confirms his work, while other points investigated were different.

This laboratory had been using, for almost two years, a certain commercial sulfur-silica compound, called herein compound "A", for capping

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‡See references at end of paper.

test specimens, when through an odd circumstance it was discovered that specimens capped with this compound did not attain the strength at ages up to 5 hr. after capping that they did when the caps were 20 to 24 hr. old. The fact was discovered when a series of relatively high-strength specimens 147 to 235 days old, were being tested. All the specimens were capped on one day, but only half were tested that day, the remainder being tested the day following. It was found that those tested the day after capping had attained an average relative strength of 123 per cent when compared with those tested the same day capped. The compressive strength of the specimens on the day capped was nominally 4800 psi.

2. DESCRIPTION OF TESTS AND MATERIALS

The observation mentioned led to a limited program in which the following features were investigated:

(a) Two sulfur-silica compounds were compared: compound "A" and compound "B", the latter being a sulfur-silica product of known high strength, which had been used in the laboratory prior to the use of compound "A". Both these sulfur-silica compounds are proprietary materials. Compound "A", as ordinarily purchased, is in the form of 10 lb. bricks, while compound "B" is purchased as a powder in 100 lbs. bags. Both materials were used to cap a series of specimens comparing two end conditions, one with plane parallel ends, and the other with the bottom end plane and the top end stepped. The step was ¼ in. deep and comprised one half of the end area. Specimens were tested at varying time intervals after capping to determine the rate of strength gain of the two materials.

(b) Compound "B" was compared with two gypsum products: one, called compound "C", was a "medium-strength" material, and the other, called compound "D", was a "high-strength" material as identified by the manufacturer. Both compounds "C" and "D" were similar in appearance and in general action to plaster-of-Paris. The effect of varying time intervals between capping and testing was studied for each material.

(c) Compounds "B", and "D" were used in determinations of the difference in apparent strength caused by variation of thickness of caps.

All the concrete tests were performed on standard 6-in. by 12-in. concrete cylinders, which were tested using a 300,000-lb. capacity Baldwin-Southwark Universal testing machine.

The concrete used for the test specimens was varied according to plan. The coarse aggregate was a ¾ in. maximum size, uncrushed siliceous gravel, and the fine aggregate was a natural siliceous sand. The

cement used complied with Federal Specification E-SS-C-191b and the water-cement ratio and cement factor were varied to attain whatever strength was desired. The slump ranged between 2 and 3 in. Each batch was mixed for 5 min. in a Smith 3 1/2S tilting mixer. A total of 252 cylinders was made and tested.

Capping with the sulfur-silica compounds was done using a Nardiello capping machine. All sulfur-silica caps were ¼ in. thick except on those cylinders which were capped with varying thickness of caps to study the effect of thickness upon the apparent strength.

The sulfur-silica compounds were heated in electric, thermostatically-controlled pots to a temperature of 248 F. ± 5F. The plaster caps were formed using squares of ⅜ in. thick plate glass pressed into the plaster to produce as thin caps as possible not exceeding 1/16 in. in all cases except, of course, where thicker caps were desired in the study of the effect of the thickness of caps. These thin caps could be made on laboratory specimens formed in molds with machined bottoms and having troweled top surfaces. Compound "C" was mixed in the proportion 1 part of water to 3 parts of "C" by weight, and compound "D" in the proportion 1 part of water to 3.57 parts of "D" by weight.

All concrete cylinders were cured in a fog room in an atmosphere of 100 per cent relative humidity and a temperature of 70 F.

3. DISCUSSION OF RESULTS

To compare the two sulfur-silica compounds, a total of 40 cylinders were made from two presumably identical batches of nominal 4500-psi. compressive-strength concrete. Half of the cylinders of each batch had plane parallel ends and half had stepped ends. Alternate cylinders from each batch were taken for capping with each compound. The twenty specimens capped with compound "A" then contained ten with plane ends and ten with stepped ends. The same was true of specimens capped with compound "B". Of each of the four groups of similar specimens, half were tested when the caps were 15 min. old and half when the caps were 2 hr. old. Relative compressive strength was based on the highest strength attained by any group. Table 1 gives the results of this series.

The stepped-end condition did not seem to have much effect on the compressive strength. There was an indication that compound "B", at an early age, gained strength somewhat faster than compound "A".

To determine the rate of strength gain for the two sulfur-silica compounds, a total of 46 specimens of nominal 3800-psi. compressive-strength concrete was made from two presumably identical batches of concrete.

TABLE 1—COMPARISON OF SULFUR-SILICA COMPOUNDS "A" AND "B" AT AGE OF CAP AND END CONDITION OF CYLINDERS BEFORE CAPPING AS SHOWN

End Condition and Type of Cap	Age of Cap, Relative Compressive Strength and Per Cent Deviation in Strength Nominal 4500-psi. Concrete 5 Cyls. each test							
	15 Min.				2 Hr.			
	Rel. Comp. Str. %	Max. Dev. %	Min. Dev. %	Av. Dev. %	Rel. Comp. Str. %	Max. Dev. %	Min. Dev. %	Av. Dev. %
Plane End, Compound "A"...	84	5.0	0.5	2.6	92	3.0	0.0	1.2
Plane End, Compound "B"...	85	4.5	1.5	2.8	100 (1)	3.5	1.5	2.2
Stepped End, Compound "A"	84	4.0	0.0	2.3	88	3.0	0.0	1.6
Stepped End, Compound "B"	88	8.0	0.5	4.2	99	4.5	0.5	2.8

(1) 100% equals 4445 psi.

All the cylinders from one batch were capped with compound "A" and all from the other batch with compound "B". Each batch of 23 cylinders was split into five groups, each group contained five cylinders except one which contained three. One group of five was tested when the caps were 1 hr. old, one when the caps were 2 hr. old, one at 18 hr., one at 24 hr., and the group of three when the caps were 40 hr. old. The 46 specimens comprising both groups were tested during one afternoon. All 46 were removed from the fog room simultaneously (48 hours before the time they were due to be tested) and remained in the laboratory air until tested. The capping was scheduled so that on the afternoon of testing some caps were 40 hours old, some 24 hours, some 18 hours, etc. This procedure insured that all specimens were alike relative to curing and drying. The highest strength attained by any group of the compound "A" specimens was with caps 40 hours old and this was considered 100 per cent relative compressive strength for that compound. For compound "B", the highest strength was attained by the 5 hour group and this was considered 100 per cent. Table 2 gives the results of this series.

It appeared from the results shown in Table 1 and Table 2 that compound "A" did not develop its strength as rapidly as compound "B". Compound "B" appeared in all cases to give specimen strengths at 1 hr. after capping approximately equal to strengths at later ages up to 40 hr.

One more set of cylinders was made for check on compound "A". These were of nominal 5000-psi. compressive strength concrete. There were 20 cylinders in the set, and they were divided into groups of five

TABLE 2—RELATIVE COMPRESSIVE STRENGTH IN PER CENT AND PER CENT DEVIATION OF NOMINAL 3800-PSI. CONCRETE CYLINDERS CAPPED WITH SULFUR-SILICA COMPOUNDS "A" AND "B" AND TESTED AT AGE OF CAP SHOWN

Type of Cap	Age of Cap																			
	1 hr.				5 hr.				18 hr.				24 hr.				40 hr.			
	Rel. Str.	Max. Dev.	Min. Dev.	Av. Dev.	Rel. Str.	Max. Dev.	Min. Dev.	Av. Dev.	Rel. Str.	Max. Dev.	Min. Dev.	Av. Dev.	Rel. Str.	Max. Dev.	Min. Dev.	Av. Dev.	Rel. Str.	Max. Dev.	Min. Dev.	Av. Dev.
Compound "A"...	83	7.0	3.0	5.0	84	6.0	2.0	3.6	93	6.0	1.0	3.2	96	7.0	1.5	4.4	100 (2)	5.0	0.5	3.2
Compound "B"...	99	2.5	0.5	1.7	100 (1)	6.0	0.5	2.8	98	5.0	1.0	2.9	97	6.0	0.5	3.4	98	1.8	0.5	1.0

(1) 100% equals 3800 psi. (2) 100% equals 3860 psi.

to be tested at 30 min., 1 hr., 2 hr., and 19 hr. after capping. All the specimens were removed from the fog room 24 hr. before the end of the testing period. The results are shown in Table 3. The strength at 19 hr. was taken as 100 per cent relative compressive strength. The relative results are also shown where the assumption is made that the strength at 19 hours was only 93 per cent of the ultimate, assuming again that the ultimate strength for compound "A" would be reached at 40 hr. (Table 2).

TABLE 3—RELATIVE COMPRESSIVE STRENGTH IN PER CENT, NOMINAL 5000-PSI. CONCRETE CAPPED WITH COMPOUND "A" AND TESTED AT AGE OF CAP SHOWN

	Age of Cap				
	30 min.	1 hr.	2 hr.	19 hr.	40 hr.
Relative Comp. Strength at 19 hr. = 100%.....	89	88	88	100 (1)	—
Relative Comp. Strength at 40 hr. = 100%.....	83	82	82	93	100

(1) 100% equals 5065 psi.

The same tendency toward delayed strength gain using compound "A" was again evident. The use of compound "A" was then discontinued by the laboratory in favor of the use of compound "B".

Compound "B" was compared with the medium-strength plaster compound called herein compound "C". A total of 44 cylinders was made from two presumably identical batches of nominal 7500-psi. com-

pressive strength concrete. Alternate cylinders were taken from each batch and grouped in fours making 11 groups for the 44 specimens. Four groups were capped with compound "B" and seven groups with compound "C". These groups were tested when the caps were of various ages. The relative strength was based on the group attaining the highest strength. The same procedure as to curing was followed with all the plaster-capped specimens as with sulfur-silica-capped specimens, that is; all the specimens were removed from fog-curing at the same time, then the designated groups were capped at the appropriate time and allowed to remain in the laboratory air until time of testing. Table 4 shows results for the compound "B" and "C" comparison.

TABLE 4—RELATIVE COMPRESSIVE STRENGTH IN PER CENT AND PER CENT DEVIATION OF NOMINAL 7500-PSI. CONCRETE CYLINDERS CAPPED WITH COMPOUNDS "B" AND "C" AND TESTED AT AGE OF CAP SHOWN

Type of Cap	Age of Cap						
	15 min.	30 min.	45 min.	1 hr.	3 hr.	20 hr.	40 hr.
Compound "B"	83	86	87	95	—	—	—
Max. Dev.	1.5	3.0	7.0	5.5	—	—	—
Min. Dev.	0.5	0.5	0.5	0.0	—	—	—
Av. Dev.	1.1	1.6	3.4	2.8	—	—	—
Compound "C"	89	93	91	93	100 (1)	99	97
Max. Dev.	19.0	4.0	6.5	3.0	3.0	3.5	3.0
Min. Dev.	1.0	1.5	1.0	0.0	0.5	1.0	1.0
Av. Dev.	9.1	2.9	3.8	1.5	1.8	1.9	1.9

(1) 100% equail 7360 pss.

The data presented in Table 4 shows that the specimens capped with compound "C" tested at 3 hr. gave the highest strength; however, the difference was not marked.

It is thought that the higher strength for compound "C" over compound "B" shown in Table 4 was due to the very thinness of the compound "C" caps. They were not over $\frac{1}{16}$ in. thick while the caps of compound "B" were $\frac{1}{4}$ in. thick. Troxell[1] shows a relative strength of 96.9 per cent for plane normal end cylinders capped with plaster-of-paris for nominal 3000-psi. concrete, and 96.6 per cent for cylinders capped with Castite, a high strength sulfur-silica compound. He also shows a relative strength of 86.8 per cent for plaster-of-paris capped cylinders of nominal 8000-psi. concrete compared to 102 per cent for Castite capped cylinders.

The strength of plaster-of-paris, according to Troxell, when tested as 2 x 4 in. cylinders is 1700-psi. at 1 hour.

Using three classes of concrete of nominal 3000, 4500, and 5500 psi. compressive strength, sulfur-silica compound "B" was compared at a cap age of 1 hr. with plaster compounds "C" and "D" at various cap ages. There was one batch of cylinders for the 3000 and 4500-psi. concrete and two batches for the 5500-psi. concrete. Alternate specimens of the 5500-psi. concrete were taken from each batch and grouped for testing. The number of specimens for each class of concrete, the number in each group and the age of cap at which tested are listed below:

TABLE 5—NUMBER OF SPECIMENS TESTED PER CLASS OF CONCRETE AT VARIOUS AGES OF CAPS FOR COMPOUNDS "B", "C", AND "D"

Strength Class of Concrete	Nom. 3000-psi.		Nom. 4500-psi.				Nom. 5500-psi.		
Age of Cap	1 hr.	3 hr.	½ hr.	1 hr.	3 hr.	40 hr.	½ hr.	1 hr.	3 hr.
Compound "B"	6	—	—	4	—	—	—	10	—
Compound "C"	—	6	—	—	—	—	4	—	10
Compound "D"	—	6	4	3	3	3	3	—	10
Total No. Specimens		18				17			37

Relative compressive strength for each class of concrete was based on the highest group strength attained within that class. The relative results for this series are set forth in Table 6.

TABLE 6—RELATIVE COMPRESSIVE STRENGTH IN PER CENT, OF CONCRETE OF NOMINAL STRENGTH SHOWN, CAPPED WITH COMPOUND "B", "C", OR "D" AND TESTED AT AGE OF CAP SHOWN

Type of Cap and Per Cent Deviation	Nominal 3000-psi.		Nominal 4500-psi.				Nominal 5500-psi.		
	Age of Cap		Age of Cap				Age of Cap		
	1 hr.	3 hr.	½ hr.	1 hr.	3 hr.	40 hr.	½ hr.	1 hr.	3 hr.
Compound "B"	98	—	—	98	—	—	—	99	—
Max. Dev.	4.0	—	—	5.0	—	—	—	3.5	—
Min. Dev.	0.0	—	—	0.5	—	—	—	0.5	—
Av. Dev.	1.5	—	—	2.9	—	—	—	2.1	—
Compound "C"	—	100(1)	—	—	—	—	97	—	99
Max. Dev.	—	6.5	—	—	—	—	2.5	—	5.0
Min. Dev.	—	0.0	—	—	—	—	0.5	—	0.5
Av. Dev.	—	3.3	—	—	—	—	1.5	—	2.8
Compound "D"	—	99	95	100(2)	100(2)	98	99	—	100(3)
Max. Dev.	—	9.0	2.0	2.0	1.0	1.0	2.0	—	6.5
Min. Dev.	—	0.0	0.5	0.5	0.5	0.5	1.0	—	0.0
Av. Dev.	—	3.3	1.0	1.7	0.7	0.7	1.5	—	2.8

(1) 100% equals 2830 psi.
(2) 100% equals 4655 psi.
(3) 100% equals 5310 psi.

From the data set forth in Table 6, it would appear that there was an insignificant difference in strength between any of the three capping materials with the classes of concrete at the age-of-cap used in the test.

To determine the difference in strength caused by different thicknesses of caps, two batches of nominal 4500-psi. compressive-strength concrete were made comprising a total of 30 cylinders. These cylinders were cast in lengths of 11½, 11 and 10½ in. When capped, all the cylinders were 12 in. long. One batch of specimens was capped with compound "B" and the other with compound "D". There were seven cylinders in the 11½ in. group, four of them being capped on both ends with caps as thin as could be made (approximately ⅛-in. for the sulfur-silica and 1/16-in. for the plaster caps) and three with caps ¼ in. thick. There were four 11-in. and four 10½-in. cylinders in each batch yielding four with ½-in. and four with ¾-in. caps for each batch. The highest strength attained by any group capped with the same compound was counted as 100 per cent. Table 7 shows the results for this phase of the work.

TABLE 7—RELATIVE COMPRESSIVE STRENGTH, PER CENT OF NOMINAL 4500-PSI. CONCRETE WITH COMPOUND "B" AND "D" CAPS OF VARYING THICKNESS

Capping Material and Per Cent Deviation from Average	Cap Thickness			
	1/16 to ⅛ in.	¼ in.	½ in.	¾ in.
Compound "B" (1 hr. old)	90	100 (1)	96	92
Max. Dev.	12.0	4.0	5.0	5.0
Min. Dev.	0.0	1.0	1.0	0.0
Av. Dev.	6.3	2.7	2.5	2.5
Compound "D" (3 hr. old)	100 (2)	99	96	96
Max. Dev.	3.0	3.0	8.0	6.0
Min. Dev.	0.0	1.0	0.0	1.0
Av. Dev.	1.5	2.0	4.2	3.0

(1) 100% equals 4745 psi.
(2) 100% equals 4440 psi.

The strength of the specimens increased as the caps became thinner for both capping materials, except in the case of the sulfur-silica caps 1/16 to ⅛-in. thick. To obtain these thin caps, the molten sulfur-silica compound was poured into a machined-out area of a steel plate and the specimen immediately pressed down into it. The steel plate was 10 in. square by ¾ in. thick and the machined-out area was 7 in. in diameter and ¼-in. deep with slightly sloping sides. It is thought that possibly some air bubbles might have been trapped under the caps contributing to lowered strength and erratic results.

Using the Nardiello capper and sulfur-silica compounds, ¼ in. was as thin as it was found practicable to make caps.

Using plate glass and the plaster compounds, caps may be made "wafer" thin, 1/16-in. or less, on a plane smooth cylinder end. It is thought that this element of thinness contributes to the merit shown by the caps of compound "C" and "D".

4. CONCLUSIONS

1. From the work done in this series it was discovered that all sulfur-silica compounds do not act alike. Compound "B" developed adequate strength within 1 hr. after capping while compound "A" continued to gain strength up to 40 hr. and possibly longer. The strength at 1 hr. to 5 hr. was approximately 84 per cent what it was at 40 hr.

2. The stepped-end condition did not appreciably alter the apparent compressive strength developed when the capping was done with either compound "A" or "B" and an appropriate age of cap.

3. For the three classes of concrete tested (nominal 3000-psi., 4500-psi., and 5500-psi., compressive strength) there seemed little to choose between compounds "B", "C" or "D". It would appear that 1 hr. is sufficient time to allow all three capping materials to harden. However, both compound "C" and "D" gave slightly higher results at an age of 3 hr. Of the two gypsum materials compared, "C" is cheaper, but somewhat less desirable to use than compound "D". "C" hardens in approximately 15 min. after the addition of water and occasionally sticks to the glass plates. "D" hardens in approximately 30 min. and no trouble was experienced with sticking.

4. Caps of either sulfur-silica compound "B" or the gypsum compounds "C" and "D" should be made as thin as possible.

5. A minimum period of 1 hr. between capping and testing appears to be essential in order that the caps may be strong enough to distribute the load evenly to the specimens.

6. Ease of operation, speed with which specimens can be capped, and the fact that compound "B" may be reused, make it the most economical and desirable of the four materials tested for capping standard 6-in. by 12-in. concrete test specimens.

LITERATURE REFERENCES

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H. F. Gonnerman, "Effect of End Condition of Cylinder on Compressive Strength of Concrete." *Proceedings*, Am. Soc. Testing Materials, V. 24, Part II, 1924.

"Sulfur for Capping Test Cylinders," ACI JOURNAL, JPP Sec., Apr. 1939; *Proceedings* V. 35, p. 420.

D. V. Terrell, "Effect of Imperfect Ends on Compressive Strength of Concrete Test Cylinders." *Engineering and Contracting*, May 1923, V. 59, p. 984.

D. V. Terrell, "Effect of Irregular Ends on Concrete Test Cylinders." *Engineering and Contracting*, Aug. 1923, V. 60, p. 228.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1945, for publication in the JOURNAL for June, 1945

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Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1944 to June 1945.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Cement Paint for Steel Water Tanks (41-158)

Q—The report, "The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces" (ACI JOURNAL, June, 1942) is full of valuable suggestions and covers the subject well as far as the title indicates; namely, application to concrete surfaces.

We encounter the question of using cement paint on metal surfaces; it has been recommended for use on the inner surface of steel tanks for drinking water such as are installed on board ships. We have not been able to find anything to answer our question: will cement paint give a durable coating on the inside of steel tanks to prevent rusting and give no taste to drinking water?

By R. B. YOUNG*

A—We have in two of our buildings hot water tanks of about 1000-gal. capacity used for supplying water for domestic purposes. These tanks were ungalvanized and it was necessary to give them some treatment which would prevent corrosion. For several years now we have been painting them annually with a portland cement and water mixture, and

*Testing Engineer Hydro-Electric Power Commission of Ontario, Toronto.

our experience would indicate that their corrosion is considerably retarded by the treatment.

A treatment that has to be repeated each year would probably not be considered a desirable coating but in this particular case the holding temperature of the water is fairly high, varying up to 160 F. Better results could be expected with tanks in which cooler water was stored. However, our general experience with paints for under water use has not shown any of them to have considerable life.

By M. B. KLOCK*

Walter Hughes of our Municipal Testing Laboratory (Rochester, N. Y.) reports that steel tanks at the Rochester garbage reduction plant are lined with gunite. The tanks are 16 ft. in diameter; the gunite was applied about ½ in. thick, reinforced with wire mesh; the surface was clean, but no special pretreatment was given to the steel. This work was done 10 or 12 years ago, and the tanks are still in good condition, which we think is a good record.

A good way to apply a portland cement paint to any surface would be by a siphon which would blow the material on under high pressure. We use such a gun here for patching concrete surfaces with mortar. (Illustrated in "Concrete in Factory Construction" by L. F. Fairchild, ACI JOURNAL, Nov.-Dec. 1934, p. 152.) In any case the steel should be thoroughly cleaned of all loose particles.

A portland cement paint is said to have been used "in the early days" to prevent or to overcome the taste of rust in water stored in metal tanks. The method was used by shipbuilders years ago for lining drinking water storage tanks. A receptacle was practically filled with whole barley grain, then filled to the top with clear warm water. When the mixture had thoroughly soured (the action could be speeded up by the addition of yeast) the water was drawn off, mixed 50-50 with clear water, and this used to mix a neat cement paint. There is a very choice variation to this: You may use stale beer instead of barley water. We tried this on a small tank and it is okay after two years.

A survey made by the Structural Bureau of the Portland Cement Assn., about 10 years ago prompted the following conclusions:

(1) Neat cement paste or a grout of cement and fine sand may be made to adhere to the inside of steel water tanks and protect them against corrosion.

(2) Whether a neat cement paste or a cement sand grout is used is largely a matter of application. The use of fine sand in the mix may tend to reduce shrinkage and therefore preserve the integrity of the coatings.

*Structural Engineer, Eastman Kodak Co., Rochester, N. Y.

(3) If the coating is always submerged or always wet, no appreciable shrinkage will occur, but rather the coating will expand and thus make cracks practically impossible.

(4) The use of admixtures that will not materially reduce the strength of the coatings and hence their bond to the steel is a matter of facility in application.

At that time the PCA acknowledged sources, chiefly railroads.

One set of specifications called for a mix of two parts of cement to one part of sand, all of which passes a No. 16 screen; the tank must be clean, the grout fresh, mixed in small batches, of a consistency of heavy cream and applied with a bristle brush that is kept clean by frequent washings. Two coats are applied, each coat being $\frac{1}{16}$ in. thick and the second following the first after 12 hours. In case the two applications do not produce a coating $\frac{1}{8}$ in. thick, a third coat is applied, allowing 12 hours time between the second and third coats. The coats are kept constantly moist from the time of application until water is admitted into the tank. This curing is accomplished by introducing a very small amount of steam into the tank.

Another source used a mix of one cement to one of fine sand applied with a brush in two coats—the second coat following the first by from four to ten hours. When the work is done from a raft it proceeds downward as the water is drawn from the tank. Ordinarily the use of scaffolding is more practical. The consistency of mix is such as to give it body and as great a thickness of coat as practicable in application with a brush. The "paint" is constantly stirred using a neat cement grout with the addition of about 5 per cent of hydrated lime to facilitate application.

One user prefers to defer painting new tanks until about one year to allow loosening of mill scale.

Still another uses neat cement grout in two brush coats about 12 hours apart, and about the same time is allowed after the second coat before placing the tank in service. A small jet of steam is allowed to enter the tank continuously while the cement grout is hardening.

"Painting Steel Portable Water Tanks" is the title of National Bureau of Standards letter circular 744 (Feb. 26, 1944) which presents a considerable digest, based on investigations and recommendations of the Army Engineer Corps, the Navy's Bureau of Ships and the New York Club of the Federation of Paint and Varnish Production Clubs. This letter covers rust inhibitors, preparation of surfaces and coatings—which do not include portland cement paint.

Long-time Shrinkage of Concrete (41-159)

By WALTER H. WHEELER*

I have been designing, supervising and sometimes also constructing reinforced concrete structures since 1908. During all of that time the problem of long-time shrinkage of concrete has been with us, and we still do not know the basic principles or mechanics of how it operates and why it operates. It causes cracks in otherwise perfect structures, which are unsightly and lead to eventual necessity for repairs. It causes permanent deflections and changes in slabs, beams and arches and is then called plastic flow. In other words it is the bane of the concrete engineer's existence.

It matters not that wood also shrinks, or that grave stone slabs which have leaned against a building for a century have been deformed by their own weight. Structural metal does not shrink and does not deform under normal stress so that it is permanently deformed.

It seems to me that we have not attacked this problem of shrinkage in concrete with the vigor and intelligence that is needed to find out just how and why it works that way and knowing that the remedies that can be applied to overcome it. It seems to me and it seems to others that I have talked with that this problem must be solved soon if reinforced concrete is to retain the high favor which has heretofore been given it, and that the cement companies should furnish the money to finance the necessary research to the extent that is necessary to solve it, and do it promptly. It is believed that this is mainly a problem for the chemist.

It also seems to me that the ACI should urge with all the energy at its command that such a research program be instituted forthwith on a scale that will get the results and get them speedily. The cement and concrete industry cannot laugh this problem off any longer.

[EDITOR'S NOTE—The gist of much correspondence on the subject discussed by Mr. Wheeler is about like this: When and if all the ills of concrete are cured, all the problems solved (and the answers appear in widely available texts), the field of concrete will cease to be interesting. Mr. Wheeler is not alone in wanting more answers to unsolved problems. Large appropriations are being invested in studies to find the answers Mr. Wheeler wants—the money supplied by those who also recognize the need for those answers. When the answers are found—or even likely clues to the answers, ACI will hope to make them known.]

*Designing & Consulting Engineer, Minneapolis

Concrete Tanks for Storage of Wine (41-160)

Q—I should like information as to the use of concrete tanks for the storage of wine.

A—Comments made by A. G. Timms, see p. 291, "Concrete tanks for wine and grapejuice?" (No. 35-27) in your JPP section of ACI JOURNAL for Feb. 1939 are still appropriate. Reference is also made to "A Comparison of Various Treatments of Cement and Steel Wine-Tank Surfaces" by W. V. Cruess, T. Scott, H. B. Smith, and L. M. Cash of the Fruit Products Division, University of California, Berkeley, Calif., in *Food Research*, 1937, Vol. 2, No. 5. The authors describe tests made at the University and among other conclusions are the following:

Treatment of the concrete surface with strong tartaric acid solution greatly reduced the amount of calcium dissolved by the wine. Two treatments, one with ½ per cent and one with 25 per cent solution gave better protection than one treatment with 25 per cent solution. A coating of insoluble calcium tartrate is formed.

Of the commercial preparations tested in these experiments Pioneer Flintkote asphalt emulsion and Bass-Hueter (made by the National Lead Co.) black enamel gave best results.

Gilsonite, 25 per cent, plus paraffin, 75 per cent, gave a highly protective coating. It is inexpensive and easily applied but is somewhat more brittle than the asphalt emulsion coating.

Precautions Against Deterioration of Concrete Exposed to Briny Air (41-161)

Q—What special precautions are advisable in reinforced concrete construction subject to the wind-driven briny air of the southeast coastal region?

A—There are many concrete structures, including exposed spandrel concrete frame buildings, along our sea coasts successfully withstanding the exposure to the wind-driven briny air. It is important, of course, to provide concrete of good quality and adequate cover over all steel. It is not only important to specify the proper thickness of concrete over the steel but also to have close supervision to see that the specifications are carried out in this respect. The following is quoted from the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete:

At those surfaces of footings and other principal structural members in which the concrete is deposited directly against the ground, metal reinforcement should have a minimum covering of 3 in. of concrete. At other surfaces of concrete exposed to the ground or to severe weathering conditions, metal reinforcement should be protected by not less than 2 in. of concrete. At undersides of slabs exposed to weather 1 in. should be provided.

Where metal fabric in the form of wire mesh or expanded metal is used as reinforcement for protective coatings on columns, beams, or girders, it should have a minimum covering of 1½ in. for structures exposed to water, ground, or weather, and a minimum of ¾ in. for structures not so exposed.

We believe that if the minimum of 2 in. of concrete is provided, as recommended by the Joint Committee, that adequate protection will be obtained. Too often considerably less than this is used and particularly over the stirrups. In many cases the specifications provide sufficient cover, but the reinforcement is not properly tied into place and maintained in position during the placing of the concrete, and consequently is too close to the surface. Steel with insufficient cover will corrode even under normal exposure but corrosion is much more rapid when exposed to moist air laden with salt.

Wire form ties should be avoided. Tie rods which are completely removed should be used or at least a type of tie rod which leaves no metal within 2 in. of the surface. Holes left by the tie rods should be carefully filled with mortar.

In making construction joints care should be taken in preparing the hardened surface and in placing the new concrete to secure good bond.

Vibrating Concrete in Test Cylinder Molds (41-162)

Q—What is the best current practice for the consolidation of concrete in test cylinder molds by means of mechanical vibration?

By H. F. GONNERMAN*

A—While a number of investigations have been conducted to determine the effects of vibration on concrete, we know of no specification or procedure recognized as standard for the consolidation of concrete test cylinders. The best rocedure in a given case will depend on a number of factors, such as the type and properties of the vibrator, the proportions, water-cement ratio, grading, consistency and other properties of the mix. The time of vibration should be just sufficient thoroughly to consolidate the entire mass, since continuing the vibration after consolidation has occurred is likely to produce segregation.

In recent years the procedure in this laboratory in consolidating concrete in 6 x 12-in. test cylinders by means of vibration has been as follows:

The mold is half-filled with concrete which is then consolidated with an internal vibrator (9,000 rpm). When consolidation of this layer is about completed and as the vibration proceeds an assistant fills the balance of the mold. The operator gradually withdraws the vibrator

*Manager of Laboratory, Portland Cement Assn., Chicago.

as consolidation takes place, care being taken to avoid leaving vibrator in mold too long as this will tend to give a core of mortar, particularly when the maximum size of coarse aggregate is 1½ in.

In a paper† giving results of tests of 6 x 12-in. vibrated concrete cylinders, Prof. M. O. Withey makes the following statement:

With the machine running at the normal speed of 3500-3600 rpm and the maximum amplitude setting, a vibration period of one minute gave best results. At speeds approximating 6500 rpm, the actual amplitude for ⅛ and ⅜ setting was less than at 5000 rpm but the time required to consolidate the concrete properly was only 20 seconds.

An illustration (p. 529) shows the arrangement of the vibrator and platform used for molding the test specimens.

The New Jersey Highway Department has made tests for compressive strength of concrete by means of vibrated specimens. The New Jersey specification for this test required the molds to be mounted firmly on a vibrating steel channel, arranged so that six can be vibrated simultaneously. The vibrator was adjusted to 3500 impulses per minute and the vibration continued for 1¼ minutes.

†"Freezing and Thawing, Permeability and Strength Tests on Vibrated Concrete Cylinders of Low Cement Content," ACI JOURNAL, May-June, 1935; *Proceedings* V. 31, p. 528.

ACI NEWS LETTER

Vol. 16 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1944

AS TO THE PROGRAM, FEBRUARY MEETINGS

41st Annual Convention—New York, Feb. 13-16, 1945

Much good material is in prospect for the ACI Convention in New York in February. Live, discussable subject-matter is in preparation, including many things of active concern to the structural engineer. But, since the Publications Committee, under the chairmanship of Douglas E. Parsons, wants to be sure that the convention grist *is* definitely a contribution to the progressive thinking of the convention audience, names and titles are not yet ready for announcement.

Not since the 1941 Convention in Washington has the Institute met in the East and not since 1939 has a convention been held in New York. The 1945 meetings should bring many of our eastern Members back into more active participation in ACI affairs.

Between now and mid-February much new War and World political history may well be written. It is not inconceivable that by February the War in the West may have reached the final clean-up stages and many of our energies be in process of reconversion to the task of overtaking our civilian needs. War accelerations of thought and act have not been without their contributions of knowledge which a peace economy will be quick to use. Certainly, construction for private and peace-time public use will be high on the list of potentials "after Germany."

Right now it can only be repeated that Tuesday, February 13 will be a day of administrative affairs—meetings of the Board of Direction and of Administrative committees.

Wednesday, February 14 will provide Technical Committees opportunity to further their tasks. To these committees we must look for correlations and interpretations that will put our scattered data from laboratory research and from field experience into terms for improved design, construction and manufacturing practice.

Thursday and Friday, February 15 and 16—and *perhaps* the evening of Wednesday the 14th—will be given over to general convention sessions.

All meetings will be at Hotel New Yorker where all those attending the ACI Convention are assured accommodations at a low maximum rate.

A New ACI Proceedings Volume is Complete with the November Supplement

Title Page, Contents, Closing Discussion and Indexes

—How to Assemble

The regular November Supplement is sent to all ACI Members and all non-member subscribers with the November JOURNAL each year. It is the answer to the scores of requests that come to the Institute every summer, autumn and winter for Title page, Contents and Index to the JOURNAL volume which is otherwise completed with the issue for the previous June.

When the JOURNALS for September and November of *last* year and the issues for January, February, April and June of this year are pulled apart (after removing the wire stitches) and the covers, "masthead" (two pages just inside the front cover of each JOURNAL issue) and the News Letter sections are all discarded, the remaining parts may be assembled with the parts of the November Supplement (after discarding its cover) to complete a new Proceedings volume.

Start the assembly with Title page and "Contents" which together make up the first section of the Supplement and follow with the first section (the first subject unit, whether paper or report) from the first issue of the volume of the previous September and so on, following the order of listed items as shown in the Table of Contents, and being careful to insert sections of discussion

immediately following the papers or reports discussed. Don't be confused by blank pages; count them as though they bore page numbers consecutive with the last numbered page of a subject unit. Blank pages are necessary for several reasons: to make possible an orderly assembly of JOURNAL subject units with subsequently published units of discussion and to provide separate prints of reports and papers from the same press run which provides the JOURNAL issues. Discussion units bear supplemental series of page numbers identifying them with the last page of the original matter discussed.

All this is a bit complicated, but it *does* meet Institute publication needs. If it is too complicated for your local bindery—or too expensive (and the Institute cannot undertake the binding of individual Member sets of JOURNALS), then it might be a good idea to do as hundreds of Members and subscribers already do: Place a "continuation order" with the Institute for its annual bound volumes. The price to non-members is $10.00 each; to ACI Members $5.00 each. The Member price is subject to a 40 percent discount (making a net price of $3.00) per volume, *provided* each volume is paid for before the first parts of that volume are manufactured, that is, *before* the publication of the September JOURNAL each year which starts a volume and all of whose parts are ready for binding only with the availability of the November supplement 14 months later.

New Members

The Board of Direction approved 41 applications for membership in August and September (36 Individuals, 3 Corporations, 1 Contributing,* 1 Junior) as follows:

Arnoldson, Carlos, Jr., P. O. Box 697, Havana, Cuba

Bansemer, Roland E., Koehring Co., 3026 W. Concordia Ave., Milwaukee 10, Wisc.

Berta, Carlos E., San Jose 1171, Montevideo, Uruguay, S. A.

Constructora General, S. A., Apartado 697, Panama, Rep. de Panama.

Coons, Kenneth Wm., Dept. of Chemical Engineering, University of Alabama, University, Ala.

Crawshaw, Arthur C., c/o Holmes & Narver, 639 S. Spring St., Los Angeles 14, Calif.

Danielson, Lloyd C., c/o J. A. Jones Construction Co., Box 299, Knoxville, Tenn.

Davis, E., "Cottesmore" 180 Rede Court Road, Rochester, Kent, England

Doanides, Peter J., Royal Greek Consulate, 30 Rockefeller Plaza, New York 20, New York.

Dole, C. Russell, Columbia Engineering Corp., 323 Plum St., Cincinnati 2, Ohio

*Florida Portland Cement Co., Box 1528, Tampa 1, Fla. (Attn. G. D. Brown)

Girlings Ferro-Concrete Co. Ltd., Great West Road, Feltham, Middlesex, England

Hatch, H. P., The Ohio Power Co., P. O. Box 137, Brilliant, Ohio

Helton, John J., Universal Atlas Cement Co., 630 Brown-Marx Bldg., Birmingham, Ala.

Hjorth, Lawrence R., 14 East 47th St., New York 17, N. Y.

Kaufer, Fred P., Varsovia No. 23—C., Mexico, D. F.

Kini, K. V., Lakshmi Bldg., Sir Pherozsha Mehta Rd., Fort, Bombay, India

Klein, Harold E., c/o Edward Ehebar Inc., 29 Meserole Ave., Brooklyn 22, N. Y.

Kloppenburg, George J., 1015 Troost Ave., Forest Park, Ill.

Lash, Stanley D., Civil Engineering Dept., Queen's University, Kingston, Ont. Canada

Lees, Henry M., 600 Benson Ave., Fox Chase, Philadelphia 11, Pa.

Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio

McElrath, Albert J., P. O. Box 29, Murray, Kentucky

Meyer, John H., 4330 Ellenwood Ave., St. Louis 16, Mo.

Millikan, Oliver H., c/o Bureau of Reclamation, Denver 2, Colo.

Ott, John E., APO 722, ASF, NWSC, Area Office, Minneapolis, Minn.

Patel, Y. G., c/o Western India Engineering Co., 45/47 Apollo St., Fort, Bombay, India

Peterson, Carlyle W., 201 Laboratory of Mechanics, Iowa State College, Ames, Iowa

Peterson, Ellis J., 1106 E. Smith St., Tucumcari, N. Mex.

Reeves, Horace A., c/o J. A. Jones Construction Co., P. O. Box 299, Knoxville, Tenn.

Roehner, T. G., 412 Greenpoint Ave., Brooklyn, N. Y.

Rogers, Franklyn C., Dept. of Civil Engineering, Rutgers University, New Brunswick, N. J.

Sami, Sabri, 23 Sabri Street, El-Daher, Cairo, Egypt

Sanchez, Dario (Vickers), Casilla 1721, Santiago, Chile, S. A.

Smith, John William, 1405 Highland Ave., Chicago 26, Ill.

Summers, William T., Jr., 2327 Cedar Ave., Long Beach 6, Calif.

Thompson, Warren H., 130 Tremaine Ave., Kenmore, N. Y.

Uncles, William J., c/o The Edward Bale Co., 171 State St., Hartford 1, Conn.

Watson, J. D., P. O. Box 299, c/o J. A. Jones Construction Co., Knoxville, Tenn.

Welton, Richard E., P. O. Box 601, Honolulu, T. H.

Young, C. B. F., c/o Clark Babbitt Ind. Inc., 630 5th Ave., New York 20, N. Y.

WHO'S WHO

Bailey Tremper

well known as a contributor to this JOURNAL, continues his pursuit of a solution of aggregate-alkali reactions in his present paper, p. 89.

E. B. Hanson, Jr., and W. T. Neelands

who record experiences with Haydite aggregates in Concrete Ship Construction (p. 105), were both with the Bureau of Reclamation until the War inspired another concrete ship program.

Mr. Hanson started with the Bureau of Reclamation in 1933 as a rodman and between then and 1942, when he went to Barrett and Hilp, concrete ship contractors, he worked on four Bureau of Reclamation Projects—The Hyrum Dam, Hyrum, Utah; The French town project, Missoula, Mont.; The Upper Snake River, in Idaho, and the Shasta Dam, Redding, Calif. From the Shasta Dam he went

to the Belair Shipyard as concrete control engineer.

On this job Mr. Neelands became assistant concrete control engineer. He had received his training with the Bureau of Reclamation starting at Grand Coulee Dam in 1936, and transferred in 1940 to Shasta Dam. His work in the Belair Shipyards began in the spring of 1943.

Thomas B. Kennedy received his degree of A.B. in Chemistry from the University of Illinois in 1930. He was employed by the U. S. Engineers in 1931 and was connected with field concrete work in lock and dam construction on the Upper Mississippi River, during the period 1934 to 1939. He has been employed in the Central Concrete Laboratory since that time as head of the section responsible for testing of aggregates, the design of concrete mixtures and the physical test of concrete specimens—whence his present contribution, p. 117.

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Feb. 1 through Oct. 25, 1944

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For further particulars address Secretary American Concrete Institute, New Center Building, Detroit 2, Mich.

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Before me, a Notary Public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily paper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912, as amended by the Act of March 3, 1933, embodied in Section 537, Postal Laws and Regulations, printed on the reverse of this form, to wit:

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Concrete Operations in the Concrete Ship Program*

By LEWIS H. TUTHILL†
Member American Concrete Institute

SYNOPSIS

This paper describes only briefly the hulls constructed in the concrete ship program of the U. S. Maritime Commission but goes into more detail in connection with problems encountered and their solution in the course of these concrete operations. Construction joint procedure, lightweight aggregate concrete and mix control, handling, placing and vibration practice, curing, testing and repair problems are described in the belief that much of this information is applicable to any concrete work of high standard. Design of hulls is not discussed except as construction is affected. The ships have not been in service sufficiently long to justify much discussion of their performance or durability.

INTRODUCTION

In the spring and summer of 1941, when demand for ship tonnage began to increase and there were indications that production of steel plate might not be sufficient for all demands, consideration was given by the U. S. Maritime Commission to the use of other materials for ships. After considerable investigation it was decided to inaugurate a program of reinforced concrete vessels. Inasmuch as such vessels require very little steel plate, their production would make use of a type of material and a class of labor which would conflict very little with the demands of steel ship construction. Construction of facilities, and later of hulls, began during 1942. While considerable difficulty was encountered during the first year of the program, due to lack of knowledge and experience, it was gradually overcome, and deliveries of concrete vessels started in 1943. The increasing number of vessels delivered in each succeeding quarter of 1943 (1, 4, 11, and 17, respectively) is evidence of the soundness of the conclusion that such ships could be built. Their performance so far, as reported, justifies the decision to build them.

*Submitted to the Institute August 22, 1944; approved for publication by U. S. Maritime Commission.
†Senior Engineer, Bureau of Reclamation, Denver, Colorado; Chief, Concrete Control Sub-Section, Technical Division, U. S. Maritime Commission from March 1943 to July 1944

During the writer's association with the program there were five concrete shipyards in operation, and five different designs of vessels built. Table 1 outlines the principal features of each design and the production of each yard:

TABLE 1—OUTLINE OF DATA ON CONCRETE HULLS

Yard	Number built	Lgth. BP ft.	Beam ft.	Depth ft.	Displacement with cargo, long tons	Cargo, long tons	Concrete cu. yd.	Steel, short tons
Savannah	7	350	54	35	10930	5430	2730	1525
Houston	4	350	54	35	10930	5430	2730	1520
Tampa	24	350	54	35	10930	5200	3100	1250
San Francisco	20	350	54	35	10930	5730	2600	1100
National City	22	360	56	38	12750	6375	3200	1655
National City	27*	265	48	17½	4000	1600	1125	490

*These lighters were an extension of the initial program.

Steel vessels of similar type and cargo capacity would require from 20 to 40 percent more steel, and practically all the steel used would be plate, instead of the relatively plentiful steel bars of the concrete vessels. While the intricate design and difficulties of construction ran the costs of the early concrete hulls very high, indications are that a properly designed concrete hull, on a production basis in well-equipped yards, could be built for no greater cost, and possibly less, than a corresponding steel hull. A good reinforced concrete vessel should have a considerably longer life and lower maintenance costs, particularly as a tanker or for other specialized cargoes, although the weight of the hull is appreciably greater. Considering all factors, it is possible that the cost per ton mile of transportation in concrete ships or barges would be no greater in some services and might even be somewhat less, than in steel ships, despite the higher cost of propelling the heavier concrete vessels, either by self-propulsion or by towing.

GENERAL DESCRIPTION OF HULLS

All vessels built during this program except the Tampa vessels were barges or lighters, with no propulsion machinery and were designed for towing, either singly or in tandem. At Tampa, self-propelled, dry-cargo vessels were built, officially called the C1-S-D1 design. The Savannah, Houston, and the first National City vessels were oil barges, composed of separate cargo holds or tanks, each of which had to be "bottle tight" on every face under a hydrostatic test with water 8 feet above the deck. Dry cargo hulls had to be practically as watertight under test, but the tests were either at lower head or made with hose pressure. The shells of the vessels ran from 4¼ in. to 6 in. thick (except the group

of lighters built at National City), contained from 3 to 5 "layers" of closely-spaced steel bars, and were designed for a coverage over the steel of ¾-in. on the exterior or outboard face, and ½-in. on the interior or inboard face throughout. It might be mentioned that our investigation of concrete vessels built during the World War I period indicated that where the coverage over the steel was ⅜ in. or more, the steel rarely rusted or became exposed by spalling of the concrete.

Fig. 1—One of four concrete oil barges built at Houston. Seven similar tankers were built at Savannah.

The oil barges built at Savannah and Houston (Fig. 1) were identical in design. These hulls had two longitudinal bulkheads and ten transverse bulkheads, the latter on 32-ft. centers, forming in the midship or parallel body section center tanks 18 ft. 4½ in. by 32 ft., and wing tanks 17 ft. 9¾ in. by 32 ft. Transverse rib framing was spaced at 10 ft. 8-in. centers and supported a system of horizontal beams at approximately 4-ft. centers which in turn supported the 4½-in. shell and 4-in. bulkhead slabs, as shown in Fig. 2 (and in Fig. 40). The bottom slab was 5 in. thick, the deck 4 in. Reinforcing details are in Fig. 3.

The oil barges built at National City, California, (Fig. 4), had only one longitudinal bulkhead at the centerline. This fact and the use of rib frames at 5 ft. 5½ in. centers as shown later in Fig. 39, without horizontal beams for the side shell (except behind fenders) resulted in an appreciably simpler structure, easier to construct than the other oil barges of the program. Transverse bulkheads were spaced at 27 ft. 2½-inch centers and were carried on a system of vertical ribs held by tie beams or struts at midheight shown later in Fig. 28. Bulkhead and sideshell slabs were 4½ to 5 in. thick, bottoms 5 in., and decks 4¾ in. Fig. 5 shows the reinforcing of these slabs.

The dry cargo, self-propelled ship built at Tampa (Fig. 6), had transverse bulkheads at 32 ft. centers with no longitudinal bulkheads except in one bay where there were two, which form two 17 ft. 5½-in. by 32-ft. wing ballast tanks with a 19 ft. 1 in. by 32-ft. void space between them.

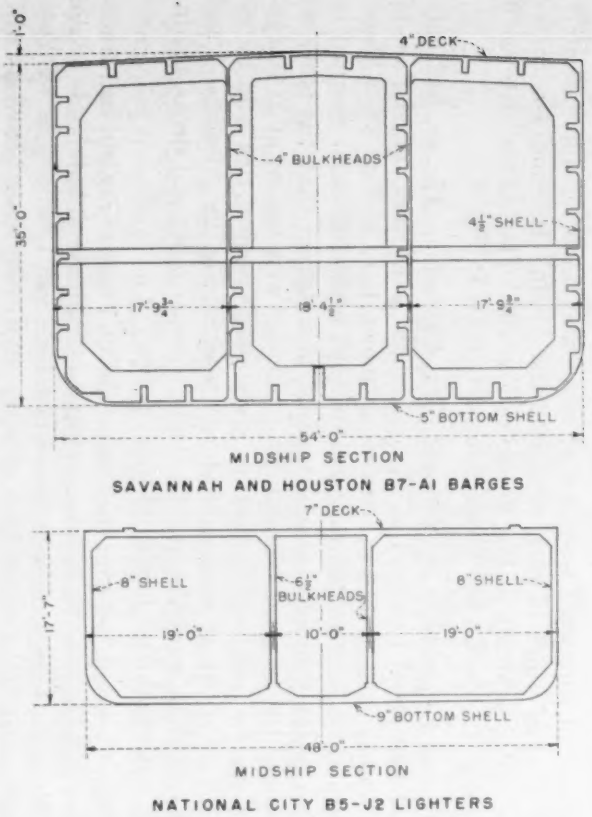

Fig. 2—Comparison of complicated and simplified design.

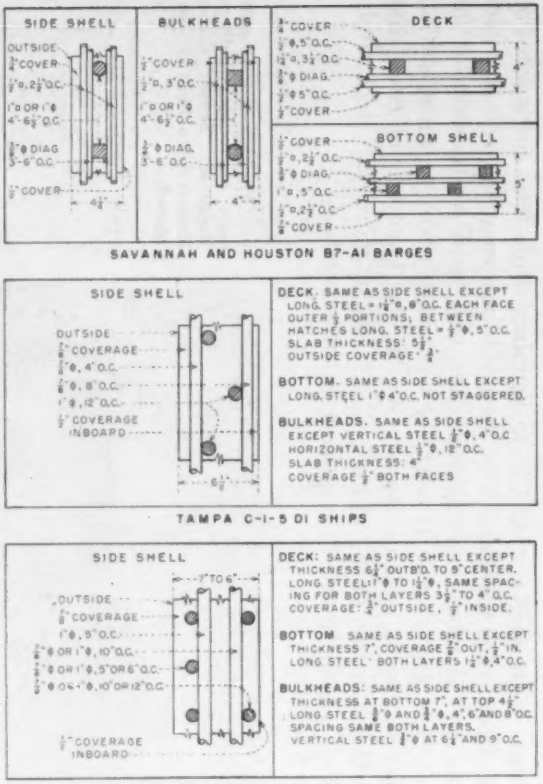

Fig. 3—Hull reinforcement details.

Fig. 2—Comparison of complicated and simplified design.

Fig. 3—Hull reinforcement details.

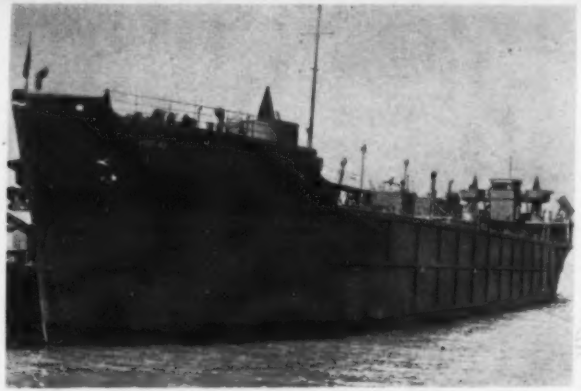

Fig. 4—One of 22 concrete oil barges built at National City, Calif.

Fig. 6—One of 24 self-propelled dry cargo concrete ships built at Tampa.

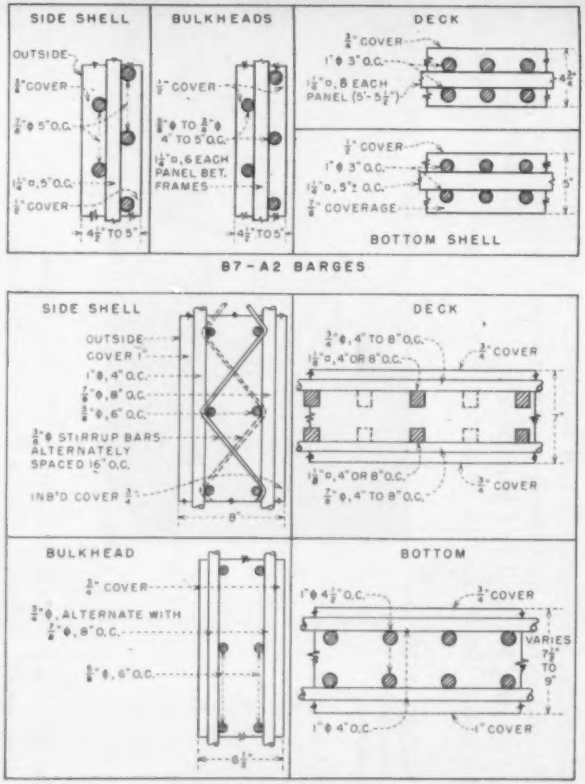

Fig. 5—Reinforcing details—National City hulls.

In these hulls, ribs on shell and bulkheads were spaced on 10 ft. 8-in. centers and carried horizontal beams at about 4-ft. centers which supported the 6½-in. shell and 4-in. bulkhead slabs. Bottom slabs were 6½ in. thick and the decks 5½ in. thick. See Fig. 3 for reinforcing details.

Fig. 7—One of 20 dry cargo towing ships built at San Francisco.

The dry cargo barge built at South San Francisco (Fig. 7) was probably the easiest hull to build in the initial program, but as Fig. 8 shows, it was by no means simple. It had no horizontal beams in the bulkheads or shell except opposite fenders on the latter, the slab of tapering thickness being supported directly on ribs at 6 ft. 4-in. centers. There were no longitudinal bulkheads. Transverse bulkheads were spaced at 32 ft. Slab thicknesses were 7 to 4½ in. bottom to top in the transverse bulkheads 7 to 6 in. bottom to top in the shell, 7 in. in the bottom, 5 to 6¼ in. in the deck. See Fig. 3 for details of reinforcing.

In frames and beams for all four of these hulls, bars up to 1¼-in. square were used in various amounts and patterns with closely spaced stirrups as shown in Fig. 9, 10 and 11. The general detail of steel in the shell, bulkhead, and deck slabs for all hulls is shown in Fig. 2 and 3. The density of steel in the concrete is realized when it is noted that about ½ ton of steel was embedded in each cubic yard.

Of entirely different design were the 25 lighters built at the National City yard in 1944. Two longitudinal bulkheads 10 ft. apart provided void spaces on the centerline as shown in Fig. 12. Six transverse bulkheads separated remaining space into 12 cargo holds approximately 19 by 48 ft. and the void space at the fore peak. This hull was unencumbered with ribs, beams, frames, columns, pilasters or struts as shown in Fig. 2. Shell sides, bottom, deck and bulkheads were flat slabs completely unrelieved except for haunches at the connections and corners. Bulkhead slabs were 6½ in. thick; all were precast (the transverse bulkheads in one piece, the longitudinals in lengths between transverse bulkheads); and in some hulls all 20 pieces, most of them weighing

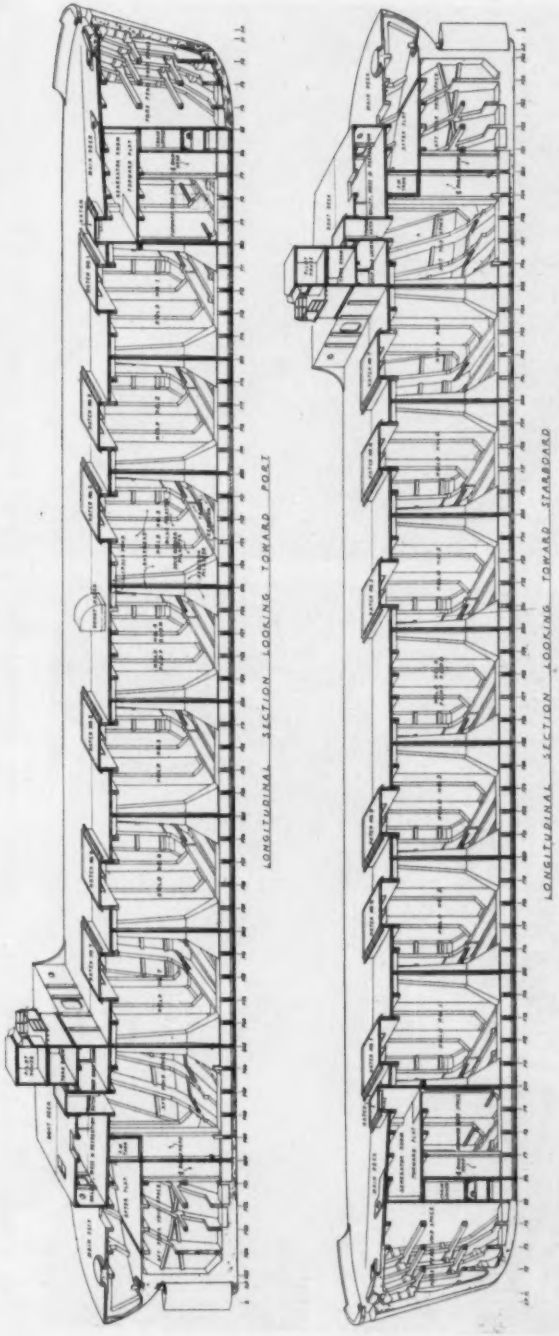

Fig. 8—Isometric longitudinal sections of B7-D1 dry cargo hulls built at San Francisco.

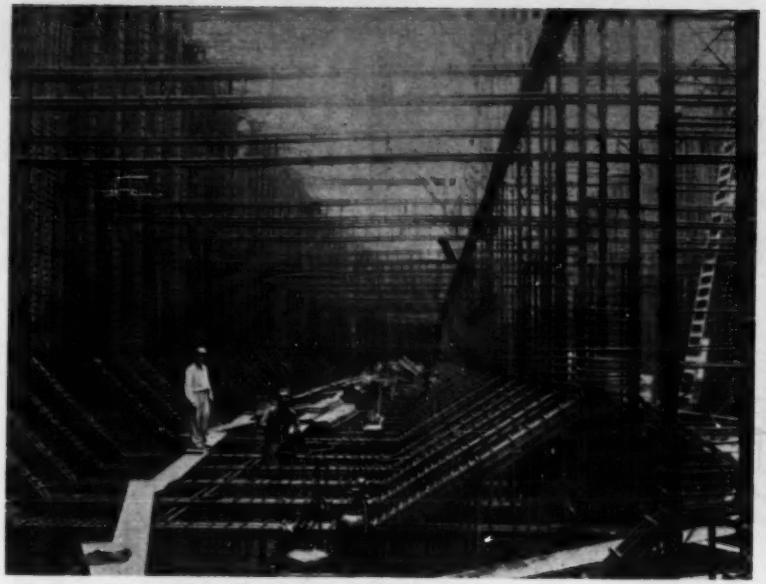

Fig. 9 (left)—Beam, haunch, and rib reinforcing in place prior to setting forms and placing first concrete in National City hull and beams to near top of haunches.

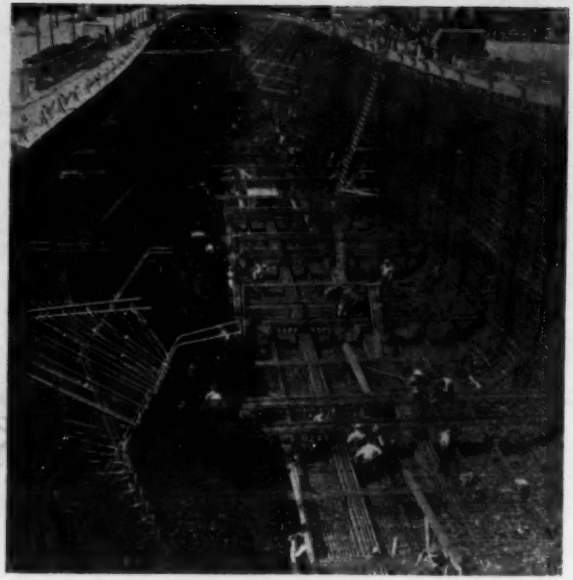

Fig. 10 (right)—Beam, bulkhead, and rib reinforcing ready for setting forms and placing first concrete in Tampa hull and beams to top of bilge.

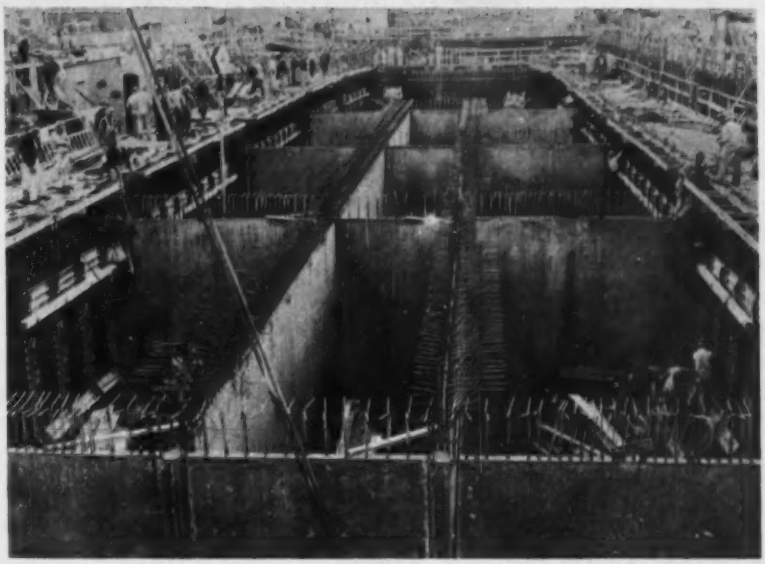

Fig. 11 (top)—Reinforcing in sideshell, beams, and ribs between bulkheads, Savannah hull.

Fig. 12 (bottom)—Precast bulkheads partially set in National City lighter. Concrete cast in bottom, sides, and deck and at intersections welded structure into a reinforced monolithic unit. One was built in 6½ days.

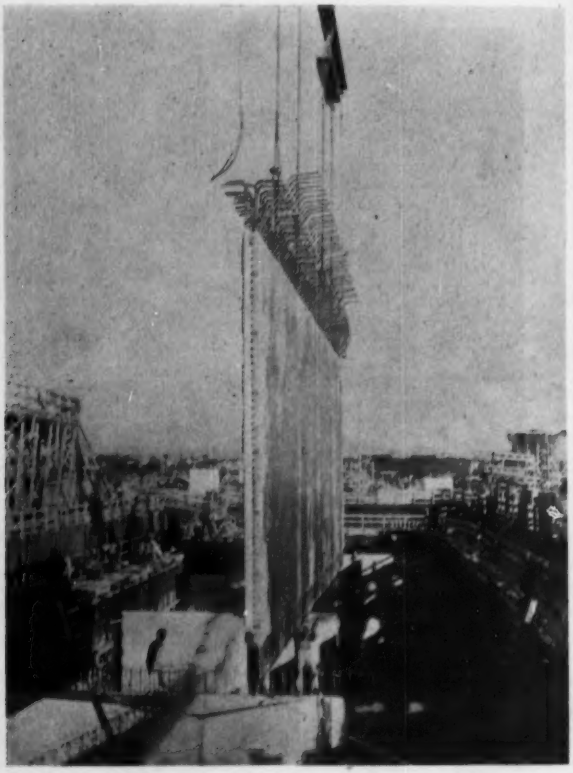

Fig. 13—Setting 22-ton precast length of longitudinal bulkhead in National City lighter. All precast bulkheads were set in this simplified hull in 3 hours.

22 to 24 tons each, were set in less than three hours (Fig. 13). There was no welding of bottom and side connections as protruding bar detail was shaped to become interlocked with steel in the bottom, side shell and deck. Longitudinal steel, $\frac{5}{8}$ in. round at 6-in. centers, protruded from both ends of the precast lengths of longitudinal bulkhead and was welded to similar bars which pierced the transverse bulkhead at the intersections. 15-in. slots where this welding was done were concreted separately before setting the deck forms. The bottom slab was $7\frac{1}{2}$ to 9 in. thick, the sides 8 in., and the deck 7 in.

With straight slab construction of these thicknesses, absence of ribs and beams, and with the reinforcing steel spaced to allow general access for the internal vibrators, these hulls were designed for efficient concrete construction. One of them was actually built and launched in $6\frac{1}{2}$ days. Because of the thicker walls and surer placing of the 2 to 3-in. slump concrete, no cracks or leaks developed in the shells during hydrostatic testing. This radically new design was far better suited to reinforced concrete than any of the earlier designs, which consisted largely of the

substitution of a reinforced concrete member for the corresponding member of a steel ship, and eliminated almost entirely the complications in steel setting, form construction, and concrete placing which caused relatively slow construction and high costs in the earlier vessels. It is a matter of regret that the idea of flat slab design without frames and beams could not have been carried out in a larger vessel, similar to the oil barges, while the program was still under way, and it is definitely the writer's opinion that any future concrete ship program should start with this type of design—or not start.

CONCRETE MATERIALS

In hulls containing about 3000 cu. yd. of concrete, nearly 40 tons of dead weight could be converted to cargo capacity by each reduction of one pound in the unit weight of the concrete. Since this multiplied to very appreciable tonnage when the 144-lb.-per-cu.-ft., natural aggregate mix was reduced 20 to 40 pounds in weight by the use of various types, amounts and conditions of light-weight aggregate, favorable consideration was given to the use of light-weight material. Tests were made to determine whether physical properties of light-weight aggregate concrete, such as flexural and compressive strength, diagonal tension, shrinkage, toughness, bond and impermeability, and other properties less clearly evaluated, were of sufficient quality to warrant acceptance of such concrete as a safe practical substitute for concrete made of natural aggregate, also whether its strength was sufficient to meet the rigid specification requirement of 5000 psi compressive strength at 28 days for hull concrete.

It appeared from these tests that light-weight aggregates could be used with confidence, but, as an additional factor of safety and to create slightly greater toughness, it was agreed to include in the mix a minimum of 15 percent natural sand. In the National City yard from the start (where light-weight fine aggregate was difficult to obtain, and expensive) and later with St. Louis coarse Haydite at Savannah and Tampa, all natural fine aggregate was used. There has been no evidence that this large percent of natural sand resulted in any measurable or observable benefit to the hulls and it did add about 400 tons extra weight. At both Savannah and Tampa no difference could be found between the hulls with the different amounts of natural sand that could be positively correlated with the difference in natural sand content.

Typical average physical properties of the various aggregates used, including the natural sands, are shown in Table 2.

Haydite, a manufactured light-weight fine and coarse aggregate, constituted the largest volume of light-weight material used. At one time or another Haydite was used in all yards and came from three plants—

TABLE 2—AVERAGE PHYSICAL PROPERTIES OF AGGREGATES USED AT THE FIVE CONCRETE SHIPYARDS

Yard	Aggregate	Aggregate Size Designation	Weight Per Cu. Ft. (dry loose) lb./c.f.	Absorption %	Bulk Specific Gravity (dry)	Percent Retained Each Size, By Weight									
						¾"	½"	⅜"	No. 4	8	16	30	50	100	P	F. M.
Savannah	E. St. Louis Haydite	½"	40	23	1.09		8	31	46	10					5	6.27
	E. St. Louis Haydite	⅜"	41	21	1.14			7	59	28					6	5.67
	E. St. Louis Haydite	No. 8	50	20	1.24					8	34	24	11	8	15	2.78
	Natural Sand	No. 8	99	1	2.62				1	3	14	31	29	16	4	2.44
Houston	Kansas City Haydite	⅝"	42	24	1.25			10	61	20	5	1	0	1	2	5.60
	Kansas City Haydite	No. 8	51	22	1.43					15	33	24	15	6	7	3.15
	Natural Sand	No. 4	101	1	2.60				9	16	13	14	36	11	1	3.11
Tampa	E. St. Louis Haydite	½"	35	12	1.12		14	30	46	5	2	0	1	0	2	6.30
	E. St. Louis Haydite	⅜"	37	15	1.14			6	60	28	4	0	0	0	2	5.58
	E. St. Louis Haydite	No. 8	46	19	1.50					7	31	23	15	11	13	2.69
	Nodulite	½"	42	8	1.29		9	41	44	2	0	0	0	1	3	6.34
	Nodulite	⅜"	46	8	1.42				44	50	2	0	1	1	2	5.25
	Nodulite	No. 4	61	7	1.86				1	15	31	17	12	11	13	2.91
	Natural Sand	No. 16	101	1	2.62						3	25	45	25	2	2.02
S. San Francisco	Kansas City Haydite	No. 8	55	21	1.49					16	38	21	11	8	6	3.25
	California Haydite	½"	46	13	1.50		13	48	33						5	6.68
	California Haydite	⅜"	47	12	1.51			2	75	18					5	5.74
	California Haydite	No. 8	65	12	1.74					29	32	17	8	7	7	3.47
	Natural Sand	No. 8	95	2	2.57				0	13	24	24	18	15	6	2.84
National City	Kansas City Haydite	½"	43	17	1.32			13	65	23						5.91
	Kansas City Haydite	⅜"	47	16	1.39			1	32	40	27					5.07
	Kansas City Haydite	No. 8	50	11	1.55					18	35	23	13	6	5	3.31
	California Haydite	¾"	44	8	1.34	1	46	34	17	2						7.27
	California Haydite	½"	48	8	1.50		7	49	42	2						6.61
	California Haydite	⅜"	49	8	1.60				79	19	2					5.77
	Rocklite	¾"	37	16	1.19	2	75	21	2							7.76
	Rocklite	½"	42	19	1.21		14	62	23	1						6.89
	Rocklite	⅜"	44	19	1.25			5	89	5	1					5.98
	Airox	¾"	39	5	1.10	3	49	38	10							7.45
	Natural Sand	No. 8	93	0.7	2.63					3	19	27	36	12	3	2.56

San Rafael, Calif., Kansas City, Mo., and East St. Louis, Ill. It is a crushed product, rough, sharp and angular. Briefly, it is made by crushing and screening the clinkered product of burning suitable shale in a rotary kiln at a temperature of about 2100 to 2000 F. until the degree of vesiculation necessary to produce material of the desired unit weight and strength is obtained.

Rocklite was manufactured in a small new plant at Ventura, Calif., and the entire output was used at the National City yard. It was produced as an individual, nearly spherical, particle with a thin shell and vesiculated interior in coarse sizes only. It was made by crushing, screening, and burning appropriate sizes in a rotary kiln at a temperature of about 2170 F. Despite the irregular shape of the particles as they enter the kiln, the processing and expansion during burning results in a remarkably well-rounded material.

Nodulite was made in a large new plant at Ellenton, Fla., and was used only at the Tampa yard. It is a coated, vesiculated, spherical particle produced by burning in a rotary kiln at about 2050 F. The

nodules were prepared for kiln feed by a "nodulizing" process in which pulverized and dried Fuller's earth was fed into a large revolving drum containing adjustable water sprays and came out rather hard damp balls ranging in size from $\frac{1}{16}$ to 1 in., which shrink considerably in burning. The nodules were dusted with fine silica sand to prevent them sticking together when burning. After burning, all oversizes and some excess intermediate size was crushed and rolled, and blended in screening with the kiln-run fine nodulite aggregate.

Type II modified cement was selected for the hull concrete because of its moderate heat of hardening (important because of the massive ribs and beams in relation to thin shell structure), its expected durability, and its better resistance to sulfate waters. For these reasons the manufacturers were encouraged to supply cements that were as far toward low heat and sulfate resisting compositions as practicable and yet give adequate strength at 10 and 28 days. Table 3 lists the general properties of cements used in the various yards. More than one column of data means that more than one brand of cement was used in that yard. Moderately high fineness was considered desirable because of the importance of reducing bleeding for the enhancement of water tightness and the benefit of workability.

MIXES AND PROPERTIES OF HULL CONCRETE

The principal mixes and their average properties are shown in Table 4. During the investigations of light weight aggregates by separate laboratories working for the Commission and for the builders, trial mixes were made and proportions arrived at for use in the yards. To secure an ample margin of strength over the required 5000 psi at 28 days and high values of tensile strength and watertightness, about 9 sacks of cement per cu. yd. were used in most mixes. With this much cement and a maximum aggregate size of $\frac{3}{8}$ to $\frac{1}{2}$ in. there were few if any mixes (at a slump from 2 to 6 in.) that were not highly workable and readily and solidly placed by the minimum amount of vibration essential to the placing of any of the mixes. To one whose experience has been mostly with relatively lean mixes and aggregates graded to comparatively large maximum sizes, all the mixes looked like mortar and extremely workable.

As is usual on a concrete job, when something goes wrong and placing results sometimes are not all they should be, even these mortar mixes would come under fire. There were solemn discussions of the significance of quarter-bag changes in cement content (3 per cent), changes of a few percent in the amount of sand or fines in the sand, minor shifts here and there in the grading curve, quarter or half-inch differences in slump. Actually there were very few imperfections, other than those due to

TABLE 3—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF TYPE II PORTLAND CEMENT USED AT THE FIVE CONCRETE SHIPBUILDING YARDS

Yard	Savannah			Houston	Tampa	S. San Francisco			National City
Portland Cement	A	B	C	D	E	F	G	H	I
Amount Tested, Bbls.	11,500	43,000	11,500	26,720	172,000*	24,000	138,000	54,500	168,500*
Chemical Analysis									
CaO	62.0	64.6	63.6	64.9	64.5	65.0	65.0	64.5	64.2
SiO_2	21.3	22.3	21.8	22.1	22.1	21.4	22.7	23.0	22.9
Al_2O_3	4.7	4.5	5.3	5.4	4.8	5.4	4.6	4.3	4.4
Fe_2O_3	4.5	3.6	4.7	4.0	4.3	4.3	3.3	3.4	2.6
MgO	4.4	1.4	1.6	0.8	1.1	1.6	1.4	1.7	2.7
SO_3	1.8	1.7	1.7	1.5	1.5	1.7	1.5	1.5	1.8
Loss on Ignition	0.8	1.2	0.6	0.7	1.2	0.6	0.6	0.8	0.8
Insoluble Residue	0.1	0.2	0.1	0.2	0.2	0.1	0.1	0.1	0.1
Al_2O_3/Fe_2O_3	1.1	1.3	1.1	1.3	1.1	1.2	1.4	1.3	1.7
3CS	46.1	49.3	46	48.6	52	55	52	48	49.
3CA	5	6	6	7.5	6	7	6.5	5.5	7
Total Alkalies Expressed as Na_2O		.16	.62	.65	.31	1.06	.57	.47	.57
Physical Tests									
Fineness Specific Surface Sq. Cm. per Gram	1850	1695	1885	1770	1860	1785	1880	1935	1855
% Passing No. 325 Sieve	87.0	83.3	91.4	92.7	87.0		92.3	93.6	93.5
Mortar Strengths									
Tensile, 1" Briquets									
3 day	290	255	275	305	275	245	300	275	270
7 day	365	355	355	390	360	350	375	370	360
Compression, 2" Cubes									
7 day		1830	2120	2775	2395	2790	2710	2680	2695
Time of Set									
Initial	3 hr. 40 min.	3 hr. 15 min.	3 hr. 25 min.	O.K.	3 hr. 20 min.	O.K.	O.K.	O.K.	4 hr. 5 min.
Final	7 hr.	7 hr.	6 hr. 40 min.	O.K.	7 hr.	O.K.	O.K.	O.K.	7 hr. 55 min.
Normal Consistency, %	23	23	23.2	24.9	23.8				
Soundness, Autoclave, %	.16	.01	.01	.19	.02	.07	.15	.04	.16

*Estimated.

*Estimated.

TABLE 4—SUMMARY AVERAGES OF TEST RESULTS OF PRIMARY CONCRETE MIXES AT FIVE CONCRETE SHIPBUILDING YARDS

Yard	Mix No.	Portland Cement, Type II	Lightweight Aggregate		Mix Proportions, % By Solid Volume					Admixture % by Wt. of Cement	No. of Cylinder Sets	Cement Content—sk./c.y.	Water Cement Ratio by Wt.	Slump—in.	Mixing Time—Min.	Unit Wt. of Fresh Concrete	Modulus of Elasticity at 28 days—10^6 psi	Compressive Strength, 6"x12" Cyl., lb. per sq. inch				Combined Sieve Anal. by Solid Vl. Percent Retained On Each Size									
			Fine	Coarse	Ntrl. Sand	Lightweight Aggregates												7 day	28 day	60 day	365 day	½"	⅜"	4	8	16	30	50	100	P	F. M.
						Fines	⅜"	½"	¾"																						
Savannah	A-1	B	E. St. Louis Haydite	E. St. Louis Haydite	15	35	50			0.5	99	9.9	.47	5.3	11	109	2.410	3760	5610	6165	7290	0	3	29	18	18	14	6	6	6	4.17
	B-1	B	" " "	" " "	25	25	50			0.5	18	9.7	.44	3.6	14	112	2.650	4590	5610	6190	7330	0	2	31	19	12	16	8	8	4	4.15
	C-1	B	———	" " "	50	—	50			0.5	14	9.5	.44	3.6	9	122	3.685	4185	6440	7215	8050	0	2	31	20	8	13	17	7	2	4.12
	C-7	C	———	" " "	50	—	30	20		0.5	31	9.5	.41	4.2	13	122	3.612	5060	6920	7380	7890	2	8	28	11	9	12	20	8	2	4.15
Houston	No. 1	D	Kansas City Haydite	Kansas City Haydite	15	45	40			0.5	85	9.6	.43	5.9	10	115	2.567	4040	5630	6235	7470[c]	0	5	28	16	18	13	12	4	4	4.22
Tampa	H-1	E	E. St. Louis Haydite	E. St. Louis Haydite	15	35	50			0.5	76	9.7	.49	6.2	7	108	2.24	4085	5875	6460	6725	0	3	26	14	12	14	13	12	6	3.75
	H-2	E	———	" " "	50	—	50			0.5	7	9.5	.46	6.4	7	120	2.62	4560	6330	6800	7150	0	4	30	13	4	13	24	10	1	3.92
	HN-1	E	———	[a]See Note	40	—	25	35		0.5	60	9.2	.42	4.1	25	117		4460	5655	6080	6585[d]	3	10	27	16	2	11	20	9	2	4.26
	N-3	E	Nodulite	Nodulite	15	35	20	30		0.5	47	9.2	.50	4.5	25	116	2.42	4205	5645	6355	7085[e]	2	14	21	15	13	10	12	7	6	4.28
S. San Francisco	AN9A	F	California Haydite	California Haydite	45	—	20	35		0.5	15	9.0	.43	6.1	3	125	3.25	3975	5850	6685	7595	1	15	24	12	14	12	9	9	4	4.36
	A	G	"	"	20	30	20	30		0.375	86	9.2	.40	4.0	3	120	3.123	4525	6355	7050	8155[f]	1	20	23	15	14	10	6	7	4	4.66
	B	G	"	"	30	20	20	30		0.375	41	9.2	.38	3.2	3	123	3.00	4485	6400	6955		1	18	26	15	14	9	8	5	4	4.69
	C	H	Kansas City Haydite	California Haydite	15	35	20	30		0.375	80	9.2	.40	4.1	3	118	2.845	4395	6140	6825		1	21	24	10	18	12	7	4	3	4.75
National City	YL-3	I	———	"	50	—	15	35			36	9.3	.46	3.1	3	126	3.62	3285	5085	5815	7280	2	17	29	4	12	12	15	5	4	4.48
	EYL-3	I	———	"	50	—	15	35		0.375	46	9.3	.46	3.5	3	128	3.55	4675	5970	6570	7350	3	19	25	5	9	11	17	6	5	4.41
	EK-3	I	———	Kansas City Haydite	52	—	14	34		0.375	193	9.75	.44	3.5	3	127	3.30	4900	6370	7095	7600	0	4	28	13	12	16	17	6	4	3.97
	ER-3	I	———	Rocklite	48	—	16	36		0.375	71	9.3	.45	3.5	3	125	3.33	5025	6330	6780	7715[h]	4	23	19	5	8	13	18	6	4	4.45

[a]Coarse lightweight Aggregate:
¾" Nodulite
½" E. St. Louis Haydite

[b] 365 day average based on 67 cylinders
[c] " " " " " " 48 "
[d] 365 day tests reported for 17 cyl. (out of a total of 31) to date

[e] 365 day tests reported for 21 cyl. (out of a total of 60) to date
[f] " " " " " 30 " " " " " 47 " "
[g] " " " " " 44 " " " " " 86 " "

leakage or failure of forms, that a little more well-timed vibration at that spot would not have eliminated regardless of the mix or the slump.

The question of consistency was much discussed in the concrete ship program. Correctly recognizing that vibration would have to be applied to every shovelful of concrete in a hull if reliable results were to be obtained, early investigators of the mixes assumed that, since such thorough vibration would obviously be necessary and, in this day of vibrated concrete, would not be impracticable of attainment, advantage could be taken of the vibration to use concrete having a slump of 2 in. Although desireable, this did not sufficiently recognize the amount of slump loss (sometimes none, often ¾ to 1 in., sometimes as high as 2 in.) that occurred from the time concrete was delivered from the mixers until it was all placed in the form, nor the human equation in obtaining complete vibration. Nevertheless, the first hull in the program was cast at National City with an average slump of 3 in. and no admixture. As in most things, the first attempt left some things to be desired and in the next hulls, average slump was increased to 4 in. where it remained for the side shell concrete but was returned after a few hulls to 3 in. for the bottom and deck concrete and excellent results were attained.

Before South San Francisco, Savannah, Houston and Tampa yards were ready to place their first concrete and prior to the writer's connection with the program, decision had been made to use a retardant admixture because some were fearful of cold joints. (Actually, reactivation of the previous lift is inconsequential, as was amply demonstrated at the construction joints. The main thing in preventing cold joints, regardless of age or hardness of previous lift, is to see that the bottom few inches of next lift is vibrated adequately.) This admixture also increased 28-day strength about 5 percent which sometimes was needed, and increased the slump from 4 to about 6 in. with the same water content. (It did not decrease slump loss however and slightly increased bleeding, even at the same slump as concrete without it.) Consequently first concrete in these yards was mixed at the 6-in. slump on the theory they needed all the slump they could get and after all, with the admixture, they were using only as much water as would be required for a 4-in. slump.

However, with the influence of the National City contractor, who fully understood and appreciated the importance of well placed medium-low-slump concrete, and who did not increase his slump when the admixture was added, and with gradual improvement of concrete control and experience in the other yards, slumps were brought down gradually until practical minimum medium slumps were generally in use throughout the program. Final average slumps were seldom much over 4 in. except when conditions were such that slump loss was consistently an

inch or more. Fig. 14 shows maximum, minimum, and average slumps for the sideshell concrete for each hull in each yard. Slumps for bottom and deck concrete averaged up to an inch lower. The South San Francisco yard was particularly successful in getting down to low slumps (Fig. 15) and at the same time getting excellent, well-placed concrete. It takes high class performance to get such results with 3-in. slump concrete as delivered at the hull or a net slump of 2¼ in. at the forms after an average loss of ¾ in.

With aging and drying of the earlier hulls and due to the various construction, testing, and natural loadings, fine hairline shrinkage and deflection cracks appeared in various beams and slabs. There was nothing abnormal or indicative of failure in these cracks. In fact, they were abnormally inconspicuous as concrete work goes. All were less than one hundredth of an inch in width; most were barely visible, probably not over two thousandths inches wide. Although of a character that would go unnoticed in an ordinary structure and the whole be rated

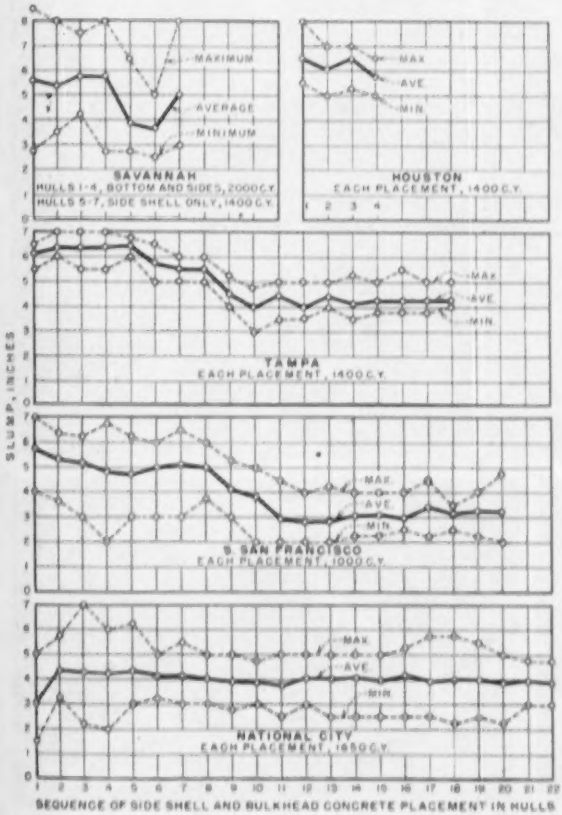

Fig. 14—Slump record of sideshell and bulkhead concrete.

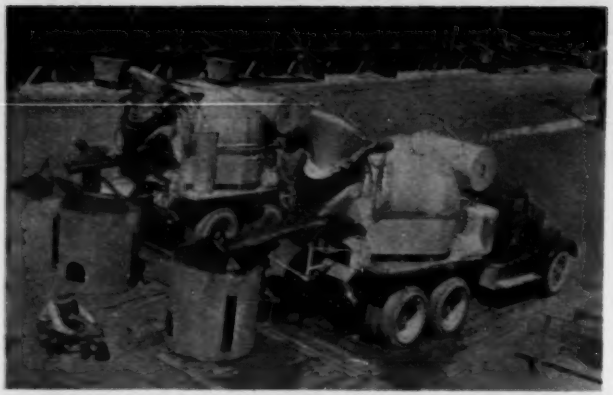

Fig. 15 (top left)—Excellent results were obtained with medium-low-slump concrete when it became generally recognized that good work depended on very thorough vibration regardless of slump. See figure 14. This concrete produced excellent results in the San Francisco hulls where best concrete work in program was done.

Fig. 16 (top right)—At Tampa a slump test was made of each 3½-yd. truck load. Note medium low slump.

Fig. 17 (bottom left)—Outside forms at all yards were arranged to pull back from the hull in full-height units. This detail was used on the San Francisco hulls.

Fig. 18 (bottom right)—Opening panel at the top of improved Tampa wall form (below horizontal beams) served to guide concrete for lower portion before closing to form upper portion.

near perfection as far as cracking goes, some of the cracking of this ordinarily negligible character caused difficulty in concrete ship delivery because in certain prescribed hydrostatic tests some of these cracks in thin panels (often due to deflection from the high load of the test pressure) permitted a slight showing of moisture and caused rejection by the ship classification society. Repeated repairing and retesting was necessary with resultant delays and the inevitable questioning of the concrete mix. Particularly the tensile strength of concrete containing Haydite light-weight, fine aggregate was questioned despite the fact that the tensile strength in concrete structural design has quite universally been regarded as zero. Consequently, since the hull mixes were little more than mortars, many briquett tensile strength tests were made and are reported in Table 5. It is considered of interest that, except for those without curing, all are substantial high values regardless of the character and amount of fine light-weight aggregate.

CONCRETE PRODUCTION AND MIX CONTROL

Aggregate was received in open cars or barges and was rehandled to stock piles and then to batcher bins by clamshell buckets or belt conveyors. The latter were less desireable and resulted in considerable separation because they necessarily deposited the material mostly at one point and there was considerable undersize material in the light-weight aggregates, especially the Haydite.

All aggregates were manually batched by weight. Most yards used bulk cement and batched it by weight. Water was measured by volume, usually in wobbly-disc water meters of standard make. There was nothing unusual about any of this equipment necessitated by use of light-weight aggregate. Some was new and some had been used previously. Automatic batching would have been preferable.

Mixing equipment varied. At the South San Francisco yard concrete was centrally mixed for 3 minutes in two 2-yd. double-cone tilting mixers and conveyed to the hull side in 4-yd. truck mixers. At National City one-yard batches were mixed 3 minutes in paving mixers at the hull. At the other three yards the concrete was entirely mixed in truck mixers a minimum of 6 to 7 minutes.

Although hourly moisture samples (and in some yards, specific gravity samples) were tested and used as a basis for batch weight and water adjustment, the basis of mix control was slump and unit weight. Regardless of aggregate tests, water was adjusted to produce the desired slump. Regardless of specific gravity tests, batch weights were adjusted on the basis of unit weight tests to produce concrete of the proper yield and cement content. Regardless of yield and computations from unit

TABLE 5—TENSILE STRENGTH, HULL CONCRETES, PSI*

Percent Natural Sand	Moist Cured		Cured 7 days, then in lab. air until 28 days old	Lab. air 28 days
	28 days**	28 days		
St. Louis Haydite concrete, Savannah mixes				
All Haydite	410	455	420	—
15% Natural sand	445	430	395	—
25% " "	470	500	450	—
50% " "	475	515	440	—
St. Louis Haydite concrete, Tampa mixes				
All Haydite	—	490	400	350
15% Natural sand	—	455	380	320
25% " "	—	600	530	430
50% " "	—	590	400	390
Kansas City Haydite concrete, Houston mixes				
All Haydite	—	505	515	310
15% Natural sand	525	550	560	350
25% " "	560	565	550	360
50% " "	545	580	560	370
San Rafael Haydite concrete, San Francisco mixes				
All Haydite	—	550	530	450
15% Natural sand	—	550	545	375
25% " "	500	540	485	475
50% " "	470	525	525	220
Nodulite concrete, Tampa mixes				
All Nodulite	430	480	433	329
15% Natural sand	435	483	428	356
25% " "	445	578	450	362
50% " "	475	529	418	362

*Each value is the average of 4 to 6 briquette tests. Sixty tensile strength tests of 20 mixes in 4½-inch dia. specimens showed an average strength 11% greater than corresponding briquette tests.
**These data from series of check tests.

weight, whole batches were measured for volume in calibrated containers of appropriate size. By means of such tests and such procedure, the mix was generally closely controlled within the limits desired. Net W/C was usually less than .50, often less than .46 by weight. Although W/C was the basis of the mix for concrete quality it was not the means but the result of mix production control.

At the beginning of concrete operations it was taken for granted that uncontrollable variations in slump and yield would result unless the highly absorptive light-weight materials were completely saturated.

When concreting started, materials had been on hand for some time and were well soaked by having been sprinkled for days previously. Naturally no difficulty was experienced with control. As concrete production increased and new supplies of light-weight aggregate arrived, there was not always time to saturate the material completely before it had to be used, as it required several days for complete saturation of the material regardless of how wet it was kept. The result was that aggregates of varying degree of saturation were used but no particular additional difficulty was noticed in maintaining slump and yield by the methods of control described above and slump loss was no more than usual.

It was then suspected that the original idea that light-weight aggregate had to be saturated for uniform results was not only incorrect but that better control and better concrete (at least lighter weight concrete) could be made if the aggregates were used in the driest practicable condition. Enterprising concrete engineers for the builder at South San Francisco completed an important series of tests of ship concretes using saturated and dry coarse aggregate and showed that compressive strength was increased slightly and unit weight was decreased about 3 lb. per cu. ft. (and remained so in wet storage for 365 days) by use of the dry materials. Contemporary investigators* found that concrete using material soaked only 30 minutes (as it would be if used dry in concrete) was at least twice as resistant to freezing and thawing as concrete using saturated material. Earlier tests† showed that drying shrinkage after 2 years was no different for concrete made of dry Haydite than for concrete made of saturated Haydite.

As this information became known in the other yards there was a marked decrease in efforts to saturate the material. Control of slump and yield on the job proved to be no more difficult than for the saturated material. Reason for this is that all the absorption that takes place at a rapid rate was completed prior to discharge of the concrete from the mixer. Consequently, when sufficient water was added to the batch to provide for the immediate absorption of the aggregate, and concrete was mixed to the proper slump before it was discharged, there was no more than usual slump loss as a result of using dry or only partially saturated material. When the material was rained on it was only partially saturated and this merely changed the amount of extra water which had to be provided for immediate absorption. From laboratory data on the absorption capacity of the material and the water content when batched, the effective W/C was computed.

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*Carlson, R. W. and Forbrich, L. R.

†F. E. Richart and J. E. Keranen, "Shrinkage of Haydite and Sand-Gravel Concrete", 1936 report of A. S. T. M. Committee C-9 (Appendix IV).

Slump and unit weight tests were frequently made for the record and as a basis of mix adjustment. At Tampa, a slump test was made on each 3½-yd. batch (Fig. 16, p. 154) but not before its discharge was begun if it appeared to be about the right slump. Sets of 6 by 12-in. cylinder specimens were made for tests at various ages. Each set represented about 250 cu. yd. of concrete. Simultaneous aggregate samples were taken from materials measured for the batch from which concrete specimens were made. The tested batch was not selected at random but was spaced at stated intervals to avoid preferential selections and permit the previous taking of aggregate samples. Typical average results are shown in Table 4.

FORMS

Except for the use of steel forms for the interior of the mid-section at National City and Tampa, forms were of wood. Sheathing of wood forms was generally ¾-in. plywood except for the bottoms of the Tampa hulls where 1 -in. T and G cypress of random widths was used.

Since all concrete placing was from inside the hulls there were no openings in the outside forms (except that in later hulls at Savannah and Tampa, there were slots at construction joints and in all bottoms for clean out). Outside forms for the entire hull were completed before any reinforcing or inside forms were placed in any of the yards. In some yards these forms were built in full height sections on towers which could be rolled back from the hull for launching (Fig. 17, p. 154). With relatively minor repair of sheathing, outside forms were reused as many as 6 times in completion of the contract and were still good for more.

Inside forms for the bottom lift were simplest of all, for the most part forming the sides of beams and ribs in which concrete could be placed through the top. Forms for the first lift in the bulkheads and the turn of the bilge usually required a closely spaced series of openings at mid height, about 2 feet above the bottom.

Forms for the second lift, shell and bulkhead walls, varied considerably for the different hulls, the principal cause of difference being the many horizontal beams in the hulls in which the transverse ribs were spaced 10 ft. 8 in. Although the open tops of these beams were admirably arranged to admit the placing and vibration of concrete, being usually about 4 feet apart, there was some concern as to the reliability of concrete placing results when attempting to place the concrete in the thin panels between beams in a 4 foot lift. This question was not answered to the satisfaction of all when forms were stripped since, in the similar hulls at Houston and at Savannah there was no great difference in results, none being perfect without certain repairs, yet at Houston the concrete was placed beam to beam while at Savannah it was placed

in shallow lifts through closely spaced openings between the beams as well as through the beams.

Inside forms for the Tampa ship, the other hull having horizontal beams, were built with the top half of the slab form between beams made in long removable panels which were placed in position only after the concrete in the lower half of the shell or bulkhead slab between two horizontal beams had been placed and vibrated. The panel was then set in and the concrete brought up to beam level (center of the beam or lower) placing through the beam. Because these forms were insecure under form vibration and resulted in bulges and overrun yardage in the hulls, they were changed to a double panel held in the center by a vertical waler bolted through the shell at each beam (Fig. 18, p. 154). In the top half of each panel was a full sized panel which opened out at the top to form a wide chute to direct the concrete into the forms to fill the lower half of the lift from beam to beam. These forms were a great improvement and reduced bulging considerably.

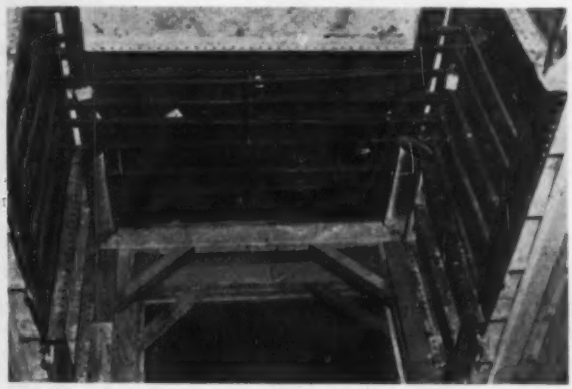


Fig. 19—Shallow-lift steel forms were placed one lift at a time as the concrete was placed at National City and provided needed accessibility for placing concrete.

From the standpoint of construction advantages, hulls with closer ribs and fewer horizontal beams were distinctly superior. In the National City hull, the only forms (steel) in place in the mid section at the start of the second lift of concrete placing were those anchored on the faces of the ribs and the first series of steel panels (12 to 15 in. high depending on location) around the bottom of the lift just above the construction joint. Successive series of U-shaped steel panels were installed after each previous series had been filled with concrete (Fig. 19). At the San Francisco yard a shutter-type wooden form was used with excellent results. The feature of this form was a tall double frame forming the panel between ribs. In the tall rectangular openings of these frames were sliding shutters, 12 in. high, each originally mounted a foot above its final position (Fig. 20). After each 12-in. layer of con-

Fig. 20 — Drop - shutter forms at San Francisco permitted 12-inch lifts of concrete with good accessibility for placing shell and bulkhead concrete. Box panels (right) were used similarly in warped and tapered surfaces.

crete was placed, another round of shutters was dropped into place and concrete raised another foot. Both types of forms afforded splendid accessibility for placing and vibrating the concrete. In this, as in all except the National City forms, frequent openings in the sides of the rib forms were provided for introduction and vibration of the concrete.

Although forms were generally as satisfactory for placing concrete as hull design would permit, and changes were made to improve their security, over-run in concrete placed (from 1 up to 10 percent of the computed volume) was never eliminated. This was a serious addition to the deadweight of the hulls and presents a challenging problem to form designers at the outset in any further construction of this kind.

HANDLING AND PLACING THE CONCRETE

There was nothing unusual about methods of transporting and handling the concrete from mixer to the forms. At Tampa mixer trucks discharged into bottom dump buckets which were swung by crawler-type cranes to buggy-loading hoppers on a runway at proper elevation for each of the 3 major placements. At the other yards, mixers discharged into skip buckets in elevators at the side of the hull and these were emptied into buggy loading hoppers at deck level and concrete was dropped through drop chutes from the buggies to the proper level or from the

buggies into position in the deck. Exceptions to the latter were for the decks at National City and South San Francisco where gantries or crawler cranes swung 1-yard buckets of concrete from the mixers to position in the deck and the concrete was dumped directly in place with some spreading from the bucket. At these two yards, location of the construction joint high, so that very little shell and bulkhead had to be placed with the deck and relatively less congestion of steel in the deck beams at these yards, made such procedure practicable where it would have resulted in seriously unsuccessful placing in the more difficult deck structures at the three eastern yards.

At Savannah, all concrete for all except the beam and slab portion of the deck was dumped from the buggies into shallow batch boxes from which it was passed by the bucketful into place, even into the bottom slab. At other yards, except in tight spots where it was impracticable to do so, the discharge of drop chutes was directed into the forms by means of small wooden chutes as the buggies were slowly dumped above in accordance with signals from a semafore at the hopper which was actuated from below. In the tight spots concrete was dumped into a batch box and shoveled or bucketed into the forms. In practically all cases, it was necessary to run an immersion type vibrator in the concrete as it entered the forms to keep it flowing out of the way from the chute or bucket.

There were many special problems in placing hull concrete that had to be recognized but their solution was largely a matter of application of basic principles of concrete placing. For instance, in early hulls poor results were obtained when the bottom concrete was started in one or both of the peaks and developed down the slope toward the mid-section. Naturally, when vibration was transmitted back through steel and forms, this concrete settled and here and there came apart and required later repair because it was not tight. This was corrected by requiring all placing to begin at the lowest point and work up the slope, thus letting transmitted vibration further compact the concrete. Even so, sometimes crews would place concrete in the bilge beams and slabs in the peaks before placing concrete below them in the keelson and adjacent flats and beams in succession toward the bilge. Trouble would result in the bottom placement when concrete was placed higher in one member, such as a transverse bulkhead, than the top of intersecting members, such as longitudinal beams, before the latter were placed and vibrated. Invariably, this resulted in pulling concrete down in the bulkhead and a repair would be necessary. This was corrected when no concrete was placed above a specified level until all adjacent placing and influencing vibration in that area was completed up to that level.

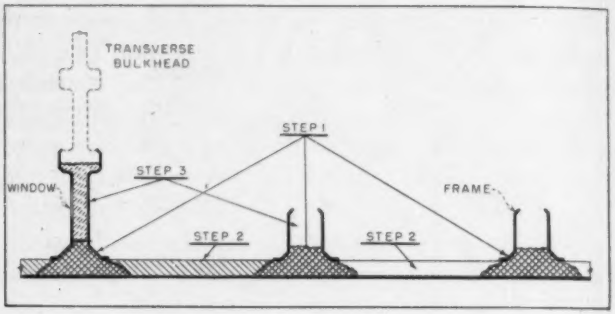


Fig. 21 — Sequence of concrete placing in hull bottoms.

The question of whether the first concrete in the bottom should be placed in the slabs or in the bottoms of beams and bulkheads was a perennial that was good for a debate anytime. Although good results could be and were produced in some cases, placing the first concrete in the slab, some reconstruction had to be done following this procedure. It was required that first concrete be placed and vibrated directly in the bottom of frame, keelson, and large longitudinal beams (but not low narrow ones which filled reliably at the bottom from the slab), and in bulkheads sufficient to fill completely the portion of the slab under such members with as little additional concrete in the members at that time as possible. See Fig. 21. This was required on the theory that good results would be best assured if the joining between slab concrete and beam concrete was made in the open rather than out of sight, under the toe plates of the form or splays, and results were good.

Height of lift came in for considerable discussion. This was not a matter for an arbitrarily selected dimension. Matters of relative accessibility for placing and vibrating the concrete and the sufficiency of vibration, required consideration as did those of form construction and the shape of the structure. There was no question but that all things being equal, a shallow lift of a foot or so (as used in the California yards) was ideal. However, the horizontal beams in the other hulls made such a lift prohibitive in form detailing. Houston elected to place the concrete beam to beam with vigorous form vibration on the panel. Savannah, in the same thin-shelled hull, cut closely spaced pockets between the beams and placed 2 or 3 shallow lifts through the pockets and 2 or 3 through the beams with the idea that frequent shallow lifts would avoid cold joints. Under the conditions, the beam-to-beam lift with good form vibration produced a tighter hull than the slow, bucket-by-bucket placing of the very shallow lifts with only such vibration as could be obtained from flexible shaft vibrators pressed intermittently against the steel and against the forms. At Tampa, the forms were built to open widely above the bottom half of each panel between beams; the top half was

placed through the beams (Fig. 18). When form vibrators were used during the placing of each of these lifts, results were excellent but, even with the thicker shell than at Houston and Savannah, the immersion-type vibrators alone could not be used to sufficient advantage to get reliable results.

VIBRATION

The success of concrete placing in all parts of all hulls depended on the thoroughness of vibration. Variations within practical limits of slump or mix were far less significant in determining the outcome of a placing operation than whether or not vibration had been adequately comprehensive and sufficiently sustained for full consolidation of the concrete. There were very few imperfections, other than due to form leakage, that a little more local vibration at the right time would not have eliminated. With reinforcing as close and congested as it was and with the inherent sluggishness of light weight aggregate concrete, 7-in. slump required as thorough (though possibly slightly less sustained) vibration as 2-in. slump concrete. Not a shovelful of concrete at any slump could be reliably placed without vibration. As these facts became more generally recognized, there was a wider recognition of the desirability of taking full advantage of this necessarily meticulous vibration regardless of slump to reduce water content of the concrete and derive the well-known improvements in concrete properties to be gained by so doing (Fig. 15, 22, 23 and 26). This is borne out by the trend of the slump test records shown in Fig. 14 and it was of interest to note that the low minimum slumps recorded resulted in no imperfections which could be attributed to them.

The question of the propriety of vibrating the reinforcing steel was often raised and never settled for some, never existed for others. At National City a device (Fig. 24) was used to attach form vibrators firmly to the steel to insure its thorough vibration in the side shell. At Savannah in their last hull, No. 7, every reasonable effort was made to vibrate only the forms and avoid vibrating the steel. Between these extremes, there was a variety of conditions in which both were vibrated but the steel received considerable vibration in all cases except in the Savannah hull 7. No difference could be found in the strength and water-tightness of the compartments of the National City hull under hydro-static test whether or not the steel was positively and directly vibrated. Somewhat lesser leakage in hull 7 than in former hulls could not be attributed entirely to avoidance of steel vibration as workmanship during placing was noticeably improved as a result of intensive instruction before concreting started. Evidently the amplitude of steel vibration

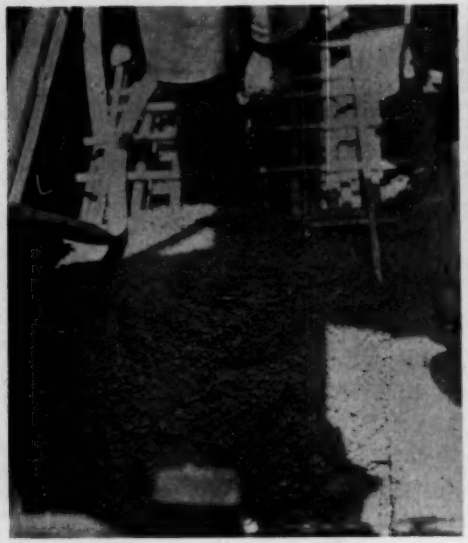
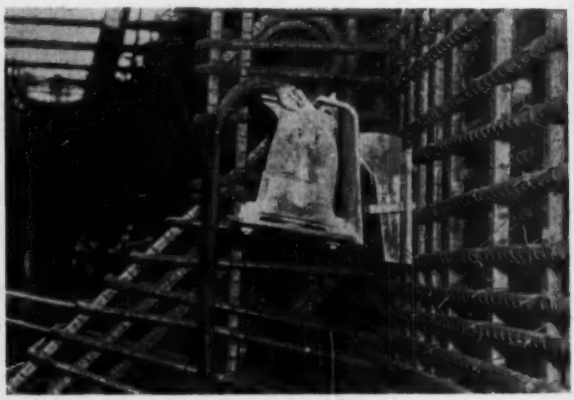


Fig. 22 (left)—Newly deposited concrete in bottom flat, Tampa (step 2, Fig. 21). Note medium-low-slump, small-maximum-size, nodulite concrete.

Fig. 23 (right)—Medium low-slump concrete, Fig. 22, responds fully to action of immersion-type vibrator, extensively used in placing hull concrete.

Fig. 24—Attachment used at National City with good results to apply form vibration directly to reinforcement during concrete placing.



was dampened and reduced so much, as it penetrated into lower lifts of partly set concrete that might be incapable of responding to vibration and again becoming plastic, that no damage to the embedment and bond of the steel resulted.

Occasionally, and usually in the vicinity of a repair needed as a result of insufficient vibration, a loose vertical bar was found. Often incomplete vibration of any kind, or wet concrete and bleeding, resulted in lack of

Fig. 25 (left)—Form vibrators were used effectively at Tampa on inside forms to supplement immersion-type vibrators which could not enter panels. Note method of direct placing from concrete buggies above.

Fig. 26 (right)—Medium-low-slump concrete placed in gunwhale beam at Tampa. Temporary pipe spreader, set with steel, maintained bar spacings for full penetration of vibrator.

tight closure under horizontal bars, particularly those of square sections. Although such bars were always tightly held by the concrete at the sides and over the bar, this condition under the bar reduced bond strength and often contributed to leakage. For this reason only round (but effectively deformed) bars should be used in horizontal positions in such highly stressed water-tight concrete construction. It can be stated unquestionably that vibration of the steel by one means and another greatly aided placing of the concrete and caused the elimination of far more imperfections than it could possibly have caused.

Best vibration, as in ordinary concrete work, was obtained from the immersion-type vibrator when it could be gotten into the concrete (Fig. 23 and 26). Because often it could not, form vibrators were used considerably. These were found difficult to reliably coordinate with placing operations when they were not used on the same side of the forms from which concrete was placed (Fig. 25). For this reason, the practice of using them outside the hulls gradually lost favor as no concrete was placed from outside the hulls.

The difficulty of concrete placing would have been much less if at the start the importance had been realized of detailing reinforcement with occasional spaces where concrete could have flowed through readily but mainly so that immersion-type vibrators could be inserted to the depth of the new concrete. As an after-thought the simplest of such

adjustments were worked out in the field (and the plans accordingly revised and approved) and the benefits in simplifying, placing and vibrating operations at these points were outstanding. But there were many other spots, mainly at intersections and in the peaks, too involved to adjust after work had started which could and should be so refined at the outset if such construction is again attempted.

CONSTRUCTION JOINTS

It was the general practice in the yards to cast the hulls in three major lifts with two construction joints, one just above the turn of the bilge, the other just under the rib haunches to the deck beams. The only exception to this was at Savannah where on the first four hulls the contractor elected to eliminate the lower of these joints but later changed to the general practice of the two joints mentioned. The lighters at National City, being so much shallower than the other hulls, were cast with only one construction joint just below the deck haunches.

At first some were concerned about the idea of having construction joints in concrete hulls but since there was ample evidence in current tests and experience that this was practicable without impairment of strength or watertightness, the practice was permitted and the problems of construction support of steel and forms were greatly simplified. In the first hull at each yard there were some imperfect lengths of construction joint which required repair. The builders quickly got this operation under control with generally excellent results.

It was soon learned that the first step in securing a good construction joint was to leave the surface of the concrete when placed well compacted and fairly even. Curing of the joint surface is important and this was continued with water until the next concrete was placed. As late as practicable after the concrete had hardened, the joint surface was treated to a full-scale sand blast of sufficient intensity to remove completely the film of surface material and fully reveal the aggregate over the entire area of the joint. Standard sand blast equipment was used. In all hulls, construction joints were made at the center of horizontal beams so that the joints would be wide enough to receive thorough treatment and not be merely the narrow width of shell thickness. Immediately prior to concreting, sand blast sand and construction debris were vigorously washed from the joint (and forms and steel above) with jets of various character from firehoses to air-water jets at 100 lb. pressure. There was an increasing appreciation of the importance of ample washout openings in the forms and some yards developed a nearly continuous opening at the joint in the outside forms.

Just ahead of concreting, the joint was coated with a soft mortar similar to that in the concrete and, as far as accessible, this was rubbed

SUCTION AND NOZZLE SIZES FOR VARIOUS USES OF SUCTION GUN					
USE TO BE MADE OF GUN	1½"-45°Y		2"-45°Y		NOTES
	SUCTION	NOZZLE	SUCTION	NOZZLE	
AIR WATER JET	¾"	¾"x 24"	¾"	1¼"x 24"	SIZES ARE STANDARD PIPE SIZES. STANDARD HOSE SIZES FOR SUCTION FIT NIPPLES OF THESE PIPE SIZES FAIRLY TIGHT. SUCTION HOSE SHOULD BE OF CONVENIENT LENGTH (6 TO 10 FT). NOZZLE LENGTHS SHOWN HAVE WORKED WELL. OTHER LENGTHS MAY BE SUITED BETTER FOR OTHER CONDITIONS. THE NOZZLE MAY BE FITTED WITH ELBOW FOR BUTTERING PERIMETER OF POURED PATCH.
DRY SAND BLAST	¾"	¾" OR 1"x 24"	¾"	1¼"x 24"	
WET SAND BLAST	¾" OR 1"	¾" OR 1"x 24"	¾"	1¼"x 24"	
PLASTIC MORT. GUN (MIX 1:1.5 PLASTIC)	¾"	¾"x 10"	¾"	1¼"x 10"	
DRY MORTAR GUN (MIX 1:4.5 DAMP)	1"	1"x 10"	2"	1½"x 10"	
VACCUM CLEANER	1½"	1½" HOSE OR TO ½" WASTE	2"	1½" HOSE OR TO 2" WASTE	

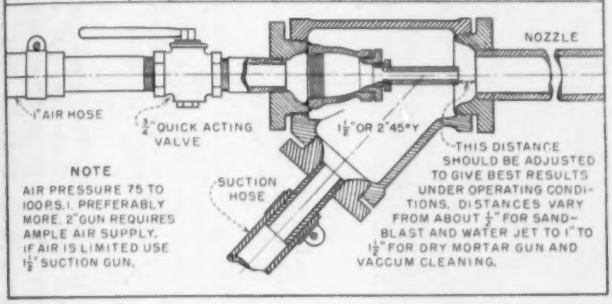

NOZZLE
1" AIR HOSE
¾" QUICK ACTING VALVE
1½" OR 2" 45°Y
THIS DISTANCE SHOULD BE ADJUSTED TO GIVE BEST RESULTS UNDER OPERATING CONDITIONS. DISTANCES VARY FROM ABOUT ½" FOR SAND-BLAST AND WATER JET TO 1" TO 1½" FOR DRY MORTAR GUN AND VACCUM CLEANING.
SUCTION HOSE
NOTE
AIR PRESSURE 75 TO 100 P.S.I. PREFERABLY MORE. 2" GUN REQUIRES AMPLE AIR SUPPLY. IF AIR IS LIMITED USE 1½" SUCTION GUN.

Fig. 27—Air - suction gun for dry sand blasting, washing, mortar application, and vacuum cleaning.

into the joint with various suitable implements. In some yards an air jet was used to spread this mortar. At Savannah an excellent job of coating the construction joint of the last hull built was obtained by shooting the plastic mortar on the joint with the air suction gun shown in Fig. 27.

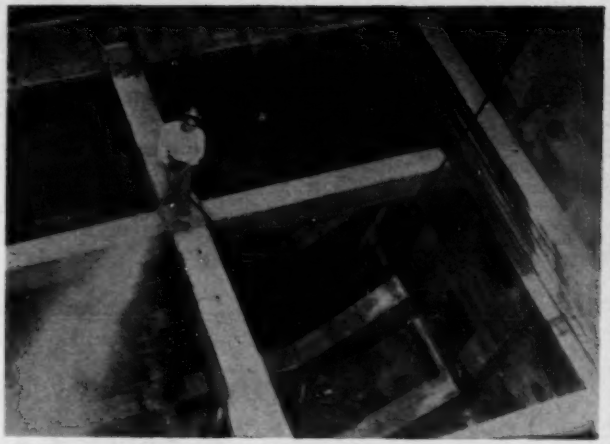
Imperfections have been due mainly to failure to remove loose material on the joint or the failure in proper vibration of the new concrete immediately above the joint, both human failures eliminated by better supervision and inspection of these operations.

CURING

Water curing, unlimited except for the vagaries of construction and when it conflicted with essential operations, was standard practice. Except for deck surfaces all early curing was from inside the hulls as in Fig. 28, as outside forms were not removed until sometime after the deck concrete was placed.

Because most published* and unpublished tests of concrete made with light weight aggregate indicated great benefit from prolonged water curing, especially in modulus of rupture and tensile strength, it was decided early in the program to extend water curing to a minimum of 30 days and, wherever practicable, maintain it until the hulls were either launched or painted.

*"The Effect of Curing Conditions of the Strength of Burnt Clay Aggregates", by W. F. Kellermann, *Public Roads*, May, 1937



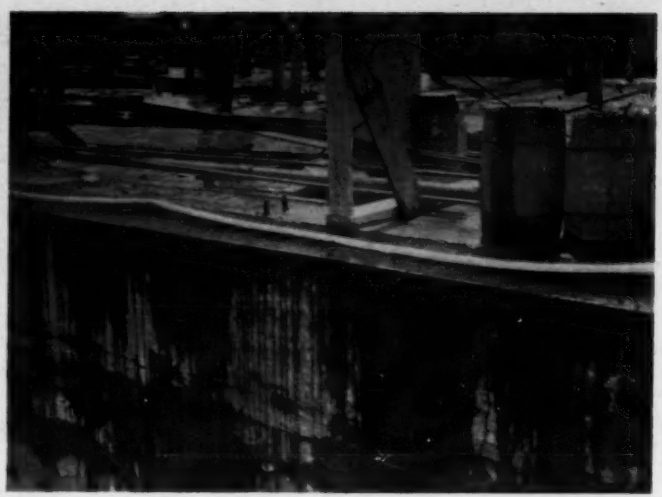


Fig. 28 (top left)—Water curing of hull interiors commenced as soon as forms were stripped. Structural details of interior National City oil barge are shown.

Fig. 29 (top right)—Canvas irrigation hose arranged along gunwhale commencing to wet outside of hull. Water quickly spreads over entire area. This was a particularly advantageous and reliable means of water curing when arranged along tops of walls, hatch coamings, and other high points.

Fig. 30 (bottom left)—Dry cargo hulls above water line were hose tested. Appearance of moisture on opposite face necessitated repair operations and retest.

Emphasis was placed on prompt commencement of curing as soon as surfaces were sufficiently hardened or were exposed by form stripping. Early stripping was urged and done where practicable as forms in place are a poor substitute for early water curing. The greatest delay came in stripping under deck surfaces because forming for another lift was not contingent on its prompt removal. Unfortunately some under deck forms were designed so that slab and sides of beams could not be stripped without removing supports from beams and this required delay. Others were designed for removal of all except the beam soffits and this permitted an earlier start of curing.

Men, and in one yard, women, with hoses were the principal means of wetting the concrete, especially the bottom lift and under the deck. Cotton "soil-soaker" hose was used to good advantage along the tops of the walls after the second major lift before deck construction progressed to the point where the hose could not be left any longer on the construction joint. Arranged around the hatches and along the edge of the deck as in Fig. 29, this type of hose was the best means used to keep deck and outside shell surfaces wet. Cotton mats were used in the southern yards for early deck curing and were excellent as long as they were kept in place. Perforated pipe was also used in some locations.

Sealing compounds were often advocated as a substitute for water curing but were not approved because (1) they were not a suitable primer for the paints and coatings to be used, (2) they might interfere with revelation of a leak under the hydrostatic tests, and (3) for the thin slab, structural concrete in hulls they would provide a curing medium too inferior to water curing to be considered favorably, particularly in view of the special need for thorough water curing of light-weight aggregate concrete.

TESTING AND REPAIR OF IMPERFECTIONS

In maritime construction, it is necessary for ships to receive a satisfactory classification by a classifying agency in order to secure favorable insurance rates and proper crew. The classifying agency designated in the Merchant Marine Act of 1920 has rules and standards based on experience with steel ships, which were the basis of its requirements in connection with design, construction and testing operations of the concrete ships. Extreme testing requirements caused a great deal of very expensive delay in deliveries. Unfortunately, there were not established, before commencing the program, classification rules which recognized any essential differences between steel plate and a thin slab of reinforced concrete.

The steel ship rules require that hull shells, liquid cargo and ballast tank bulkheads, decks, chain lockers and fuel and water tanks be absolutely watertight under hydrostatic pressures of as much as 8 ft. above the deck for periods of at least four hours. Dry-cargo hulls must be tested by examination for leakage inside with the hull immersed 18 to 20 ft. and by hose test above that water line (Fig. 30). In the hose test a fire hose with a pressure at a nozzle of 30 psi is played for an hour on each 100 sq. ft. of hull surface from a distance of 10 feet and no moisture may show on the opposite side for the hull to be acceptable.

Fig. 31—Typical minor leaks resulting from high hydrostatic test pressures with hull tanks filled with water under head 8 ft. above the deck. Even leaks like small damp spot at left of the larger spot were required by the classifying agency to be repaired, supposedly through the shell.

The classification rules make no distinction as to what is a leak requiring repair and what is not. Pressure was held on each tank a minimum of 4 hours and any appearance of water was marked as a leak and repair was required by the classifying agency even though it was only a small damp spot as in Fig. 31. It was believed by many that the classification rules placed an unnecessary handicap on construction of concrete vessels because of the requirement that such leaks be repaired through and through, necessitating as much work to make them tight and dry as leaks obviously requiring repair. Insignificant and inconsequential leaks in concrete would never get larger but would probably get smaller and seal off altogether. On the other hand, in steel, for which the rules were written, such leaks often could be stopped with one or two blows of a caulking tool and should be stopped, because in steel, experience shows they would never get smaller but probably get larger due to working of the hull. Thousands of man hours were spent and many days of hull service were lost working on scores of small damp spots which, all together, as one workman said "Would not make a gallon of water all the way to Australia, and most of them would be dried up before it got there." Bilge pumps serve only as standby equipment on a concrete ship and are never used to pump water leaking into the hulls except as a

result of accident. The hulls are dry and dusty inside. Sweating is almost entirely absent, in marked contrast to reported sweating in steel hulls under some conditions sufficient to start the bilge pumps.

As the program progressed and workmanship became better in placing the concrete, testing difficulties decreased. It became more generally recognized that well-vibrated concrete of medium-low slump was more nearly watertight than wet-placed concrete, probably because reduced bleeding and less stratification resulted in a continuity of solids that meant positive watertightness even in the 4½-in. walls under a 40-foot hydrostatic test head. Furthermore, after a seemingly endless exhibition of individualism in all the tanker yards relative to repair methods standardized practice for repairs was established which was based on methods that were sound in principle and well proved in practice on the job.

A large item in repair procedure was the "poured patch" for reconstruction of areas either unfilled originally or so poorly consolidated as to require replacement to secure watertightness. Many a spot showing slight permeability was adequate and sound structurally but, under the severe hydrostatic test and watertightness standards, it proved cheaper and quicker to remove and replace such areas as soon as they were located than attempt to seal them by grouting methods, often unsuccessful and always very time consuming.

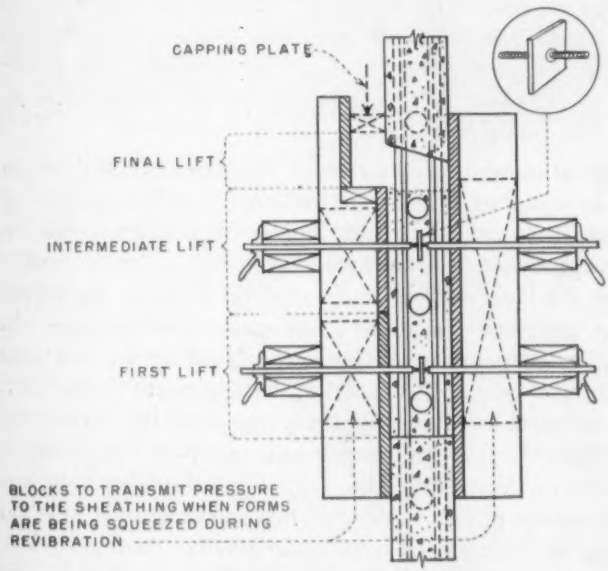

CAPPING PLATE
FINAL LIFT
INTERMEDIATE LIFT
FIRST LIFT
BLOCKS TO TRANSMIT PRESSURE TO THE SHEATHING WHEN FORMS ARE BEING SQUEEZED DURING REVIBRATION

Fig. 32—Forms and fastenings for concrete replacement.

Fig. 32 shows a squeeze-type form which invariably produced tight replacements when established procedure was carefully followed. Briefly,

the hole was trimmed out with air-driven chipping tools making sure that all concrete that would leak was removed, that there were no sharp corners but that edges were squarely cut except for the slope to the access chimney at the top, that steel around the perimeter of the hole was ½ in. clear of the concrete, that the contact surface of the concrete was kept wet at least overnight and sandblasted (see Fig. 27, 35 and 36) and thoroughly washed just before the form and new concrete were placed. The front form for larger holes was made in one-foot lifts and the edges of the holes to be covered in each lift were buttered with soft mortar of the same mix as in the concrete, well rubbed in just before the form and concrete were put in place. Concrete was the same as the hull mix without admixture but as dry as could be vibrated solidly into the forms. It was found that 3-in. slump was about right for the first lift and that for succeeding lifts, by the time mortar and forms had been placed, concrete from the same batch, though less in slump, was still readily vibrated solidly in place. Such premixed concrete, though of very low slump by the time the top of the replacement was made, could with care be fully molded into place with vibration and had little or no settlement. As late as the concrete would still respond to vibration, the concrete in place was vibrated through the forms while simultaneously the top center form bolt and the chimney cap wedges were tightened (Fig. 33). Vibration stopped when the tightenings stopped. The complete enclosed and restraining forms of these recast areas presented an ideal opportunity to take advantage of a small amount of aluminum powder admixture to prevent settlement before setting and insure watertightness of the upper perimeter of the repair, but approval of its use was withheld by the classifying agency.

There was an understandable reluctance to cut through and recast an area containing an isolated damp spot or leak but the classifying agency required repair that would be continuous from inlet to outlet, regardless of how insignificant the channel might be. This naturally led to grouting, little of which was successful mainly because the percolation channels were too small to receive the grout and because the cement did not stay in suspension in the majority of grout mixtures used. Where the channels were not too small, grouting made them tight; where they were too small, those made tight were made so by the surface treatment (hereafter described) at the source and outlet of the channel. With water on one side of the wall a 1-in. deep 1-in. diameter hole was drilled with a star drill at the point of issue. When the water was lowered an air pressure testing bell was placed over these holes in turn and their entrance points located by bubbles in a soap solution applied to the opposite side of the wall. A 1-in. hole was similarly drilled at these spots. By means of an impact grout gun (see Fig. 34) several shots of

grout were applied at each of these holes both sides of the wall and after proper cleaning the holes were solidly dry packed. Some were made dry by driving a tight wooden peg in the hole after filling it with soft cement paste. Many a time the tiny hole was plugged by the caulking action during drilling the 1-in. grout hole.

Fig. 33—After completion of concrete replacement, forms were revibrated before the concrete set and form bolts and pressure cap wedges were tightened. See Fig. 32.

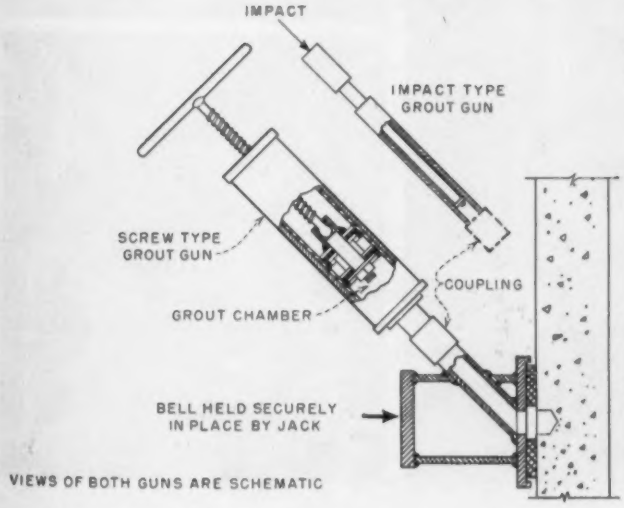


Fig. 34—Screw - type and impact-type grout guns.

Fig. 35 (top left)—The air-suction gun (Fig. 27) was used for wet or dry sandblast (depending on condition of sand in bucket) cleaning of cutouts for repairs.

Fig. 36 (top right)—Following sandblasting, cutout surfaces were washed with an air-water jet using the air-suction gun (Fig. 27).

Fig. 37 (bottom left)—Premixed dry-pack mortar, shot in place in shallow imperfections with the air-suction gun (Fig. 27), made well-bonded sound repairs.

Fig. 38 (bottom right)—Repairs were best cured by fastening wet burlap mats against them.

Late in the program at the Savannah yard a screw type grout gun (see Fig. 34) was placed in operation and better suspension of cement in the grout was obtained by first dissolving one percent of a bentonite product in the grout mixing water. Using 2.5 to 3.0 parts of this mixing water to 1 part of cement by weight, there was a marked improvement in results from grouting operations. In most cases water from the grout appeared at the other end of the channels, giving good assurance of penetration and filling well into these small passages. Occasionally, recognizable grout would come through. Areas so treated had few damp spots on refilling the tanks. The thoroughness of this type of grouting for individual wet spots resulted in tacit approval of grouting repair work from only one side with appreciable saving in time and cost. Even this improved grouting was unsatisfactory for repair of cracks and wet perimeters of poorly constructed concrete replacements, as wet spots commonly occurred on retest between the grout holes which were only 4 to 6 in. apart.

Testing pressures severely loaded the hulls and the weight of testing water severely loaded the slightly yielding ways. Result was that deflections caused hairline cracks to appear which showed some dampness. Repair was required by the classification agency despite the fact (proved in many instances) that, with the extreme test load removed, the cracks were closed tight, sealed themselves, and showed not the slightest dampness under ordinary service-head conditions. These were repaired by cutting a deep, narrow slot along the crack on each side of the wall using a saw-tooth bit in the chipping gun and, after an overnight period of moistening and a thorough sand blasting and washing, solidly filling them with dry packed material slightly wetter than usual but placed in half-inch layers with an interval between to avoid a "rubbery" condition that resulted in a loosened bond.

There were various shallow areas requiring repair where no leak or damp spot was involved. After proper trimming, wetting and cleaning, these were replaced either with premixed mortar, layer by layer, with a stiffening period between each layer, or with the mortar guns (see Fig. 27) which applied with excellent bond and density a ready mixed 1:4.5 natural or light weight sand mortar of a very dry consistency but little wetter than for dry pack. See Fig. 35, 36, 37, 38.

In the latter part of the program this mortar gun replaced trowelled repairs and dry pack including work on damp deflection cracks and damp spots. For replacement with the mortar gun it was necessary to flare the cuts to avoid inclusion of rebound. To keep rebound minimum and avoid an over-rich replacement it was necessary to avoid too dry a replacement. Plugging of the gun prevented the material from being placed too wet. Two-layer work and careful trimming later, before it

set, prevented damage to bond. Finishing by rubbing with a rag without adding water was advocated in preference to wetting and trowelling.

Gunite was tried prior to adoption of the mortar gun and found unsatisfactory for this work mainly because it was too large a facility for the purpose. Neat work could not be done in the small areas involved; extensive, expensive cleanup of surrounding surfaces was required later because the cleanup was seldom promptly done; attempts made to use gunite for through holes and to stop leaks resulted in a low percentage of successful repairs.

PAINTING

All concrete hulls were required to be painted above the light load waterline, with the standard "low-visibility gray" color selected by the Maritime Commission for war-time operation. Concrete is somewhat similar to this in color, but not sufficiently close to satisfy the requirements. It was decided that hull bottoms need not be painted, as the concrete is resistant to seawater, and the steel was well covered. No anti-fouling paint was required as information at hand indicated somewhat less barnacle formation on concrete than on steel, and the long period of service before repainting would be possible, would make the original application of little benefit. Some contractors elected to paint the bottoms, however, and were permitted to do so.

Bottom surfaces and sides below the light load line were faired by bevelling offsets 1 inch for each ⅛-in. thickness of the offset. Airholes were filled by sacking, on both painted and unpainted concrete, on the hull exteriors, but unpainted concrete surfaces on interiors, such as dry cargo holds, ballast tanks, and storage flats, received no surface treatment after form removal and necessary repairs had been made.

Interiors of holds designed for oil cargo were coated, either with vinylite resin paint (Fig. 39) or with an emulsion of Thiokol latex reinforced with Osnaberg fabric (Fig. 40). Where gasoline was carried this was necessary to protect the gasoline from contact with the concrete, since the alkaline nature of the concrete causes a reaction which breaks down the gum-inhibitors in the gasoline.

Clean, formed surfaces required little or no preparation for application of the Thiokol. Only a clean, dust-free, hard concrete surface is required. Offsets must be roughly bevelled, but no sacking or stoning is required, even for pinholes. No acid etching is required. Where laitance had formed, or cement washes had been used, sandblast proved the most efficient means of removal. The Thiokol latex coating seemed the only surface treatment of the concrete which was sufficiently elastic to bridge and stop leakage of any slight deformation cracking.

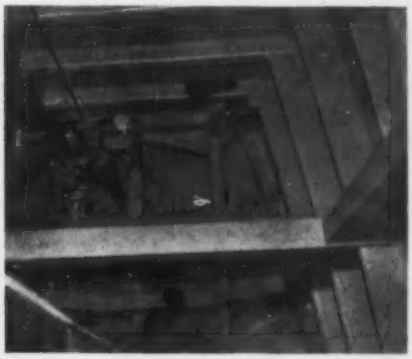


Fig. 39 (left)—Vinylite resin paint (black) was used on interior of cargo tanks in the National City oil barges.

Fig. 40 (above)—Thiokol latex (white) reinforced with Osnaberg fabric was used on interior of cargo tanks in the Houston and Savannah oil barges.

Other paints, both for exterior and interior, required surfaces which had been made smooth by a flexible-disc, rotary, electric sander, and then sacked to fill air bubbles and irregularities. It was necessary to etch the surfaces with a 20 percent or stronger solution of muriatic acid. Where the acid was applied by spray instead of brush, a 50 percent solution was found necessary. After brooming or agitating, the acid wash was removed with water so that there would be no powdery bloom when the surfaces dried. For all coatings except the Thiokol the concrete surface had to be dry.

Paints of the vinylite resin base type have given the best service to date of the available materials, and were used for both exterior and interior surfaces, including gasoline cargo holds. The Thiokol latex coating has apparently performed well also.

The writer is indebted to A. D. Kahn, Senior Materials Engineer, for assistance in reviewing and editing this paper, and to Pell Kangas, Associate Materials Engineer, for preparing the drawings and assembling the tables. Messrs. Kahn and Kangas were employed in the Concrete Control Section of the U. S. Maritime Commission during the construction of the concrete vessels.

Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1945 for publication in the JOURNAL for June 1945.

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Discussion of a paper by Lewis H. Tuthill:

Concrete Operations in the Concrete Ship Program*

By L. COFF and AUTHOR

ERRATA

Attention is directed to three errors which should be corrected in the paper. (1) In Figure 2 the shell thickness of the Savannah and Houston hull should be 4¼ inches instead of 4½ inches. (2) On page 142, last paragraph, first line, the number of lighters should be 27 instead of 25. (3) In Figure 20 box panels are shown at the left of the figure, not (right) as the title states.

By L. COFF†

Mr. Tuthill's paper, "Concrete Operations in the Concrete Ship Program," supplies official data concerning concrete hulls of recent construction. Details are given regarding the time of preparation and construction and the methods employed for the building of 77 ships and 27 lighters. Cost data would have been very welcome, also information regarding navigability of these ships.

When comparing the hulls described by Mr. Tuthill with the work of the Emergency Fleet Corporation, and concrete shipbuilding generally during and shortly after the World War I it occurs that the hulls dealt with in the paper do not differ much in design and methods of erection from the concrete ships built over twenty years ago. It would be difficult to find an analogy in other engineering fields for such a lack of progress. Were these boats so perfect as to defy the engineering skill of the next generation?

The report of the ACI Special Committee for Reinforced Concrete Ships and Barges, *Proceedings*, V. 17, p. 285, 1921, contains a very lucid summary of concrete shipbuilding activities and results of the preceding

*A.C.I. Journal, Jan. 1945; *Proceedings* V. 41, p. 137.
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period. The following disadvantages of concrete ships, built and in service, were mentioned.

1. The proportion of weight of hull to total displacement was unfavorable as compared with steel ships. The committee mentioned that a .55 to .60 efficiency factor—proportion of cargo to total displacement—was expected, whereas for the actual boats the hull with equipment weighed 50 percent of the total displacement. It is mentioned in the report that for a steel ship of similar characteristics, the efficiency would be .65 to .70.

The data given by Mr. Tuthill show no progress in this respect while, in the meantime, the efficiency factor of steel ships has decidedly improved through welding.

2. The committee states that the time of construction was generally longer than that of steel ships.

There is no need emphasizing that this drawback is more marked now than it was then. Steel shipbuilding beat all records as to speed in the recent emergency by adopting revolutionary methods. These were evolved in order to allow welding in the most favorable and comfortable position. The same considerations could possibly have been applied to concrete. While horizontal concreting is easy, and favorable to mass production, vertical concreting of narrow webs, between forms crammed with steel rods, has proved in the last emergency to be difficult, slow and inefficient.

3. The committee dealt with the difficult navigability of concrete ships, emphasizing their lack of resistance to impact. The statement is made, that it was difficult to secure the best class of sailors for concrete ships on this score. From the drawings produced in Mr. Tuthill's article, it cannot be inferred that these shortcomings have been eliminated or even improved upon.

4. As to the cost of concrete ships, the committee found that they were excessive. A price of $200. per ton dead weight was the basis of the estimates of the types dealt with, while the final figure after construction was close to $280. This meant no savings as against the cost of steel ships.

It would be interesting to compare the cost of the concrete boats dealt with by Mr. Tuthill with the cost of steel ships built recently.

When considering the above, the question arises in everybody's mind who, like the writer, has spent years of his professional life in designing, constructing and assisting in the operation of concrete ships, why the mistakes of the first emergency had to be religiously repeated and ships built which will be the first to be laid up, after the emergency.

Would not concrete seem an ideal mass production material, which could have relieved the shipping situation substantially, if other methods than the one proved ineffective had been used?

The reply to this question would express the feeling of frustration of a number of concrete boat engineers of 20 years ago, who thanked God when the concrete boat came up again, for getting a chance to devote to the war effort, a seemingly useless experience acquired through efforts full of agonies. The worst of them was to have to fight the mental attitude of the classification companies.

In the former emergency their surveyors, distinguished and experienced naval architects, bought their first textbooks on reinforced concrete the very day they started putting down the law and telling the concrete engineer what to do and what to leave alone. Reinforced concrete was then less than twenty years old as an industrial proposition. Supervisors of concrete boat construction are probably now more familiar with the material, than they were then.

However, what cannot be changed, as results prove, is the intrinsic contradiction betweeen a pioneering job of considerable magnitude and the functions that classification companies have efficiently discharged through several centuries, i.e., to protect insurance companies by rationing changes in shipbuilding, on strength of experience and experience only.

No concrete boats had been built since 1922. What more natural than that classification companies should insist on starting where we left off then?

However, concrete ships are implements of war. Classification companies are neither called on to insure submarines nor bombers, nor even transport planes. If concrete ships were desirable for alleviating the shortage, why ignore past experience and not give the concrete industry a free hand as to design and erection, on a competitive basis as wide as compatible with safety?

It is the contention of the writer and of many others in the industry that, given sufficient freedom, concrete engineers and contractors could have produced a serviceable, quickly built and reasonably priced concrete ship. Some people in the profession even believe that, for certain specialities, the concrete boat could have become a permanent institution. Adapting factory floor design to shipshape, proved a mistake, so why repeat it?

A start was made in August, 1941, when a competition for tanker design with bids was called by the Maritime Commission. It had an enormous response but nothing came of it.

On strength of the experience in the last emergency, the desire to obtain insurance for crew, ship and cargo must have been the deciding factor against departing from obsolete methods in concrete boat construction.

The concrete industry did not fail; it simply was not given the chance to do an appropriate job.

AUTHOR'S CLOSURE

Mr. Coff has raised some pertinent questions. On many of these, complete information was not available when the paper was written early in 1944. Others were not within the scope of the title.

It is true and admittedly regrettable that the four designs for the 77 ships built in five yards did not differ very much from the designs for the concrete ships built at the time of World War I. As Mr. Coff has correctly surmised, the classifying agency had much to do with this due to its refusal at the start of the program to permit any departure from previous standards and concepts of design because there was no time to prove the many new factors involved. Simplified designs of slightly less favorable deadweight ratio were considered initially but were abandoned in favor of proven conventional rib and shell design of better efficiency, despite the handicap placed on both time and cost of construction.

The suggestion was frequently heard during the program that since concrete ships are, as Mr. Coff points out, emergency implements of war, the restraints and delays resulting from classification might well have been dispensed with in favor of simplified design and test requirements, faster construction and delivery, and lower costs. As explained in the paper, this was prevented by the Maritime Act of 1920 which requires classification for all ships built by the Maritime Commission. Consequently the concrete ship program was automatically burdened with classification and the many resulting needless and impractical requirements that well could have been ignored in producing safe, practical hulls of high utility in the emergency.

It should not be overlooked, however, as perhaps Mr. Coff has, that a great forward step was taken toward the end of the program in the design and construction procedure for the 27 lighters built at National City and described in the paper, pages 142 to 147 and in Fig. 2, 5, 12 and 13. One of these was built and launched in 6½ days, a record equaled in the construction of few steel ships 265 feet in length. The precast construction of bulkhead walls in a horizontal position did much on these hulls to answer Mr. Coff's objection to "vertical concreting of narrow webs."

While final cost figures are not yet available for the various yards, on the basis of the preliminary figures which have been prepared, it appears that the costs of the concrete ships in this program were not out of line when properly compared with the costs of the first steel ships produced in new yards by builders inexperienced in ship construction. The Houston and Savannah yards were closed before their high costs on early hulls could be reduced on later ones. But at National City the cost of the rugged oil barges that have served the Navy so well in the Pacific was reduced from about $300 to $135 per ton of cargo capacity in the course of construction of the 22 built. At San Francisco the cost of the dry cargo barge dropped from about $193 to $101 per ton in 20 hulls. The self-propelled ship at Tampa had, as was known from the start, a comparatively unfavorable deadweight ratio and unit costs were consequently high. During construction of 24, costs dropped only from about $2,125,000 to $1,630,000.

Reports to date on performance of the concrete ships and barges have been consistently favorable. They have handled well, even in the most turbulent seas. They have been sturdy and resistant to fire and damage from close explosions. The self-propelled ships are remarkably free of vibration. All the hulls are dry and there is very little condensation. A cargo of Canadian wheat loaded in bulk against the hull concrete arrived in San Francisco without a kernel being sprouted or any grains being swelled or stuck together. Seaworthiness was demonstrated by the splendid performance of the S.S. Aspdin as it headed into the hurricane off Cape Hatteras, Sept. 14, 1944. According to the captain's report the ship "Behaved admirably throughout the hurricane. She pitched somewhat, but there was no rolling, panting, weaving, or pounding. As a result of her fine behavior, there were no injuries to any of the officers or crew—not even a scratch. The wind was estimated at 120 miles an hour, and the waves from 50 to 100 feet in height." In other severe storms the various barges and lighters showed equally admirable seaworthiness. On all hulls crews quarters were comfortable and well furnished. After becoming acquainted with the special characteristics (and the idea) of a concrete ship, sailors have generally preferred to stay with the ships built in this program.

In collision, the thin slabs of the rib-beam-and-slab-designed hulls are somewhat vulnerable but, in the several occasions where bumps of various force have occurred, the structural frame has not been damaged and repair was a relatively simple matter of recasting the panel in the manner described in the paper. Following the tests* at the National Bureau of Standards which demonstrated 1½ to 3 times greater resistance to impact with slabs with "supplementary reinforcement in the

*"Impact Resistance of Reinforced Concrete Slabs" by R. W. Kluge, ACI Journal, April, 1943, page 397.

form of overlapping helices of large diameter wire, commonly known as spirals," the builder of the Tampa ships proposed the spirals be used and the Commission approved after investigation showed they did not interfer with the placing of adequately vibrated concrete. The spirals had been installed in the shell of the first hull when the classification society objected to their use and required their removal on the technicality that the spirals were slightly large and therefore encroached a short distance into the required coverage of the reinforcement. The interwoven design of zig-zag stirrups (Fig. 5 of the paper) used in the shell of the lighters probably accomplished much the same purpose. Combined with the thicker wall, these features of the advanced and simplified lighter design did much to eliminate the weakness in impact attributed to the concrete hull.

In addition to the improvement in design for concrete ships that was developed for the lighters toward the end of this program, there were other improvements in comparison with the earlier program. Among the items which contributed to superiority of the hull structures built in the recent program are vibration as an aid in placing concrete, improved procedures for the preparation and bonding of construction joints and for the repair of imperfections, better cements, effective concrete control procedures, considerable welding of reinforcing steel, and probably better curing and painting.

Many who were connected with the recent program will appreciate Mr. Coff's comments on the "classification companies." In fairness, however, it should be recorded that the surveyors in this program were construction men generally familiar with proper conduct of concrete operations. Within the restrictions of their instructions, as individuals they were cooperative and helpful and contributed appreciably to the inspection and control of the work.

In some informal discussion there has been an indication of misunderstanding concerning the parenthetical statement about cold joints on page 152. The point was made that if this statement is true the treatment of construction joints described on page 166 was unnecessary. This inference is incorrect because it ignores the unescapable reality of the adherent contamination which invariably accumulates on the surface of construction joints, especially when several days or weeks elapse before new concrete is placed. Obviously this adherence must be removed, preferably with sandblast and as late as possible to avoid further contamination, before the new concrete is placed, if a well-bonded joint is to be obtained. The statement would have been more specific had it included the words "if it is clean and free of contamination" so the statement would have read "The main thing in preventing cold joints, regardless of age or hardness of the previous lift if it is clean and free of

contamination is to see that the bottom few inches of the next lift is vibrated adequately." However, within the time during a concreting operation that a new surface may have to wait for the addition of concrete, no contamination is likely to occur that will detract from the joint bond if the first of the new concrete is well vibrated. If sufficient laitance develops to interfer with such a bond, other action is badly needed. There was practically no laitance from the ship concrete due to the high cement content and medium low slump.

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Fully and Partly Prestressed Reinforced Concrete*

By PAUL WILLIAM ABELES†
Member American Concrete Institute

SYNOPSIS

In this paper it is intended to show the distinction between fully and effectively partly pre-stressed concrete, employing high strength steel. The various systems and methods are described, "pre-stretching" and "post-stretching" being distinguished. The losses of the initial pre-stress are discussed and formula are derived for the factor of safety against cracking and for the minimum stretching force to ensure "full" prestressing, to be reduced for "partial" prestressing to such extent that dangerous cracks are avoided. Published comparative test results of prestressed and conventionally reinforced beams are discussed and some new data are presented regarding preliminary comparative tests on beams reinforced with high tensile wire. It is shown that a partly prestressed beam, having a reinforcement of about one fifth of that required for mild steel, behaves similarly to a conventionally reinforced beam, when under working load, and similarly to a fully prestressed beam (i.e. remaining crackless), when the load is removed.

1. INTRODUCTION

In the design of reinforced concrete it is a standard rule to base the design on a "cracked" section independent of whether cracks occur in the construction or not. The width of cracks varies with the percentage of reinforcement and the working stress of the steel, depending on the bond effected. In order to avoid the danger of corrosion to the reinforcement, which can be done if the width of cracks is limited to, say, 0.01 in.[1],[2]‡, the working stresses are limited to 25,000 to 34,000 psi.

Prestressing, generally, denotes that the reinforcement is tensioned *before* the *load* is applied, the stretching force being transmitted as compression to the concrete after the latter has attained sufficient strength to take up the stresses occurring at this stage. By this process stresses are imparted to the structure of opposite sign to those occurring under load.

*Received by the Institute April 17, 1944.
†Consulting Engineer, London.
‡See references at end of paper.

Pre-stretching and *post-stretching* indicate whether the tensioning is carried out *before* or *after hardening* of the concrete. With pre-stretching the products have to remain in the molds until the stretching, produced by tensioning the reinforcement against anchorages at its ends, can safely be transmitted to the concrete, which is done mainly by bond. There is one exception only, when prestressed reinforcing units with self-contained compression members according to H. Schorer[3],[4] are used. In this case the molds can be removed before the concrete has hardened, the connections and compression members of the reinforcement being withdrawn after the concrete has attained the required strength. The self-contained compression is thus transmitted to the concrete by virtue of the bond.

Post-stretching is carried out against the hardened body of the concrete. The molds can be removed soon after production but special provisions at the ends of the reinforcement are necessary for the transmission of the compression to the concrete, there being no bond between the reinforcement and the concrete. There is an exception when the steel is electrically prestressed due to the increase of temperature* according to K. P. Billner[5], in which case the bond is restored afterwards.

With pre-stretching at the release of the stretching force on the concrete, the initial prestress is immediately reduced owing to the elastic deformation of the concrete and to shrinkage, which losses gradually increase later by further shrinkage and plastic flow of the concrete. With post-stretching no immediate losses, owing to the elastic deformation of the concrete and to the first part of shrinkage, occur.

2. THE VARIOUS SYSTEMS OF PRESTRESSING

Table 1 contains a summary, distinguishing various systems on the basis of their first publication either in articles or in letters patent and referring to the progressively different purposes for which the systems were evolved.

Already at the very beginning of reinforced concrete, prestressing was used to strengthen the structure by tightening the reinforcement but without considering the degree of prestress. In his 50 year-old Patent "Constructions of Artificial Stone and Concrete Pavements", P. H. Jackson, of San Francisco, (1)† described many methods for performing the stretching of ties, provided in the footings (impost beams) of arches along their length by skewbacks, turnbuckles, screws and nuts, wedges etc. In the year 1888 another patent was applied for, which might be related to prestressing.‡ Short members of triangular section for use as

*The method of tensioning by heat has already been suggested by the Czechoslovakian Boh. Ruml (U. S. Patent No. 2,061,105).

†The numbers in brackets relate to the single cases in Table 1.

‡German Patent No. 53,548 granted 1890 to C.F.W. Doehring.

fireproof protection of timber floors are manufactured from mortar and tensioned wires.* The method suggested for stretching a number of wires simultaneously appears rather interesting.

The idea of counteracting the straining due to loading by prestressing was for the first time clearly expressed by the Austrian J. Mandl (2*a*), who wanted as early as 1896 to utilize the strength of the concrete as much as possible by reducing the concrete tensile stresses under load. A formula for the magnitude of the stretching force in accordance with this proposal was derived by the German M. Koenen (2*b*) in 1907 to the end that the concrete tensile stress in a triangular straight line stress distribution, obtained for working load, should be limited to a certain magnitude. The Norwegian J. G. F. Lund (2*c*) suggested the production of straight vaults, consisting of rows of prefabricated "brittle" blocks, jointed in mortar, with prestressed tie rods arranged between the rows in a wide mortar joint. The compression is transmitted to the blocks by washer plates at the ends, and the bond is destroyed at stretching. Similarly the American G. R. Steiner (2*d*) proposed to tighten the reinforcing rods first against the green concrete, thus destroying the bond, and to increase the tension after hardening of the concrete. The last two proposals thus represent the first steps towards effective "post-stretching." However, in all these cases the losses owing to shrinkage and plastic flow were not considered; the stretching force was therefore too small and tests did not, in consequence, give satisfactory results. It can safely be assumed that the initial prestress in all these propositions was less than 18,000 psi. in order to meet the existing regulations regarding permissible steel stresses.

A further development of prestressing led to the demand for guaranteed absence of cracks, which can be ensured reliably by "*full*" prestressing; in this case the stretching force has to be of such a magnitude that in the sections of the structure, when under working load, even after all possible shrinkage and plastic flow has taken place, concrete tensile stresses are eliminated. R. H. Dill, of Alexandria Nebr., (3*a*), appears to have been the first, in 1923-25, to make such a proposal to be carried out by "post-stretching." In this system a bonding between concrete and reinforcement is prevented by coating the latter with a yielding substance, and the stretching process is carried out after a great part of the shrinkage has taken place, thus largely avoiding the losses of the prestresses, which rendered the proposals described above, ineffective. Dill's idea was a process "that recognizes the difference in qualities of concrete and steel and combines the two materials in a scientific manner",

*The explanation given by Doehring does not, however, appear to meet the point by stating that if an extensible material such as wire under a primar tension is combined with a second, inextensible material, such as mortar, both materials, when under tension due to load, will be strained together nearly to the same extent up to their ultimate strength, the breaking of both materials occurring simultaneously; nothing is mentioned of tightening, counter-action or pre-compression.

TABLE 1.—THE VARIOUS SYSTEMS OF PRESTRESSING FOR STRUCTURES, STRAINED BY BENDING.

No.	Purpose of Prestressing	Process Suggested	Name of Proposer	First Publication		Pre-Stretching	Post-Stretching	Way of Achievement of the Purpose
				Year	(Place—time of issue)			
1	Strengthening the structure	Tightening to a not determinate degree.	P. H. Jackson San Francisco	1888	U. S. Pat. No. 375,999 (1888)	yes	no	Many methods for stretching the reinforcement.
2	Reduction of the concrete tensile stress to a given limit and reduction of cracking	Counteraction due to stretching the tensile reinforcement, being not or only partly effective since initial prestress too small and losses not or only partly considered.	a J. Mandl Vienna	1896	Journal Austr, Ass. Eng. & Arch. (Z.d.Oe.I.&A.V.)	yes	no	Stretching before concreting.
			b M. Koenen Berlin	1907	Central-Journal Germ. Bldg. Ass. (Zentral Bl. d. D. Bau Verw.)			Stretching by hydraulic jacks before concreting.
			c J. G. E. Lund Bjorn	1907	U.S. Pat. No. 1,020,578 (1912)	no	yes	Rods having threads tightened by nuts between prefabr. blocks.
			d C. R. Steiner Gridley-Cel., Cal.	1908	U.S. Pat. No. 903,909 (1908)			Rods having threads tightened by nuts against green concr. and afterwards stretched again.
3	Guaranteed cracklessness.	Counteraction by full prestressing, the stretching force being of such magnitude that no tensile stress occurs when under working load, considering the greatest possible losses of the prestress.	a Rich, H. Dill Alexandria Nebr.	1923 1925	U.S. Pat. No. 1,684,663 (1928)	no	yes	Destruction of bond by coating.
			b W. H. Hewett Minneapolis Min.	1927	U.S. Pat. No. 1,818,254 (1931)			Similar to Dill's proposition.
			c E. Freyssinet Neuilly-sur Seine, France	1928	U.S. Pat. No. 2,080,074 (1937)	yes	no	High strength steel or wire stretched, reinforcement substantially reduced.

				d	Thom. E. Nichols Hornell, N. Y.	1931	U.S. Pat. No. 2,035,977 (1936)			Tensile reinforcement in excess of usual requirements.
				e	F. O. Anderegg, Newark, Ohio	1934	U.S. Pat. No. 2,075,633 (1937)	no	yes	Tensioned tie rods extending through perforated ceramic blocks.
4	Extended applicability (Increase of span).		Tensioned ties in combination with normal reinforced concrete.	a	F. Dischinger Berlin	1934	Brit. Pat. No. 464,361 (1937)	no	yes	Ties, hanging in curved lines, engage externally the reinforced concrete elements.
				b	U. Finsterwalder, Berlin	1936	U.S. Pat. No. 2,155,121 (1939)			
5	Reduction of cracking (similar to 2 but effective).		Partial counteraction by combination of an effectively stretched and an unstretched tensile reinforcement.		F. Emperger Vienna	1939	U.S. Pat. No. 2,255,022 (1941)	yes	no	Unstretched main reinforcement in usual manner and bonded additional prestressed rods of superior strength.
6	Saving Steel	a	Partial prestressing.		P. W. Abeles	1940	Brit. Pat. No. 541,835 (1941)	yes	yes	The tensile reinforcement substantially reduced (thus differing from 5) by the use of high strength steel or wire also for the unstretched reinforcement.
		b	Partial counteraction by partial pre-stressing high strength steel.		London	1942	Brit. Pat. No. 554,693 (1943)			As before but the whole reinforcement prestressed to the same or to different extent.

permiting a "crackless concrete" which is "free from tension". A further advantage exists according to Dill "in utilizing hard steel or steel of high elastic limit and high ultimate strength" instead of mild steel. As far as the method is concerned "nuts and rods" are suggested as the "cheapest way of stretching." Threading the hard steel would present some difficulty; but Dill does not limit his methods to nuts and refers to any other possibility of stretching.[6] W. H. Hewett, of Minneapolis Min., (3*b*) made suggestions similar to those of Dill, after he had successfully applied his idea to circular tanks in 1922[7], in which latter case the structure was not strained by bending but by tension only.

Independent of these American ideas, the French engineer, E. Freyssinet (3*c*) put forward in 1928 his scheme of creating from concrete a new homogeneous material using high strength steel or wire bonded to the concrete. This allowed such a high prestress that after the losses mentioned the residual stretching force was great enough to exert permanent compressive stresses in the concrete sections of the structure when under working load; moreover, a great saving of steel was achieved.* This idea was further developed in Germany by Hoyer, who introduced the use of super high strength wire (piano or string wire) and suggested the prefabrication of articles in a long continuous run, later cutting them into pieces of the required length, which is only possible with a homogeneous material.[8]

Another proposal for full prestressing is that of the American T. E. Nichols, of Hornell, N. Y., (3*d*) who suggested a bonded reinforcement "substantially, in excess of that warranted by prior practice" (i.e. mild steel). This proposition, however, would not appear to be very suitable (since a substantially greater reinforcement is required than usual), save for the purpose of ensuring absence of cracks, which could be attained by the previously mentioned systems with a great saving of steel.

The quick assembly of high strength ceramic blocks to form a fully prestressed structural unit was suggested by the American, F. O. Anderegg, of Newark, Ohio, (3*e*). He used tie rods, preferably of high strength steel, extending through perforations of the blocks. The interspaces between the prestressed tie rods and blocks are filled with a rather liquid mortar, no safe bond being ensured between the post-stretched tie rods and the blocks.

The idea of post-stretching ties in arched bridges with a suspended carriage originates from the German, F. Dishinger (1934); he (4*a*) and U. Finsterwalder (4*b*) (who had also suggested post-stretching the tension members in lattice girders) applied this idea to beams in order to extend the applicability of reinforced concrete structures to long

*Another suggestion of E. Freyssinet relates to a method of post-stretching by using a jack for tensioning and wedging (U. S. Pat. No. 2,270,240).

girders and bridges, where normal constructions are not so suitable. In this case curved ties engage externally conventionally reinforced concrete elements which may be divided into several sections at the joints. The tensioning is performed either by the dead weight only of the whole structure, without ensuring cracklessness, or by stretching, which may be carried out to such an extent that total absence of cracks is guaranteed.

Another aim is to increase the strength and to reduce the formation of cracks to a desired extent by adding to a conventional main reinforcement, carrying in the main dead and live load, an additional prestressed bonded reinforcement of higher quality steel, the prestress, however, although effective, being moderate. The suggestion of the Austrian engineer, F. V. Emperger (5) is a development of aim (2) of Mandl with the special purpose of permitting the permissible stresses for conventional reinforcement to be increased in view of the increased resistance against cracking, without using high strength steel as the main reinforcement.

Saving steel is the main aim of the writer's proposals. In one of them (6*a*), similar to the suggestion of Emperger, a combination of prestressed and unstretched reinforcement is employed, but the whole reinforcement is substantially reduced by using also for the unstretched reinforcement high strength steel, preferably wire. Both pre-stretching and post-stretching are applicable. In the other suggestion (6*b*) the whole of the high strength reinforcement is tensioned to the same or a different extent, either the whole or a part of the reinforcement being prestretched.

3. FULL AND PARTIAL PRESTRESSING

Whereas systems 1 and 2 in Table 1, constituting ineffective methods of prestressing, are mainly of historical interest only, system 3 relates to full prestressing and systems 5 and 6 to partial prestressing, system 4 allowing of each of these processes. The use of high strength steel reinforcement is suggested in systems 3a, 3c, 3e and 6, with the possibility of a substantial saving of steel. With system 3 of "full" prestressing the conventional methods of calculation can be dispensed with, since the sections remain under permanent compression. No objections can therefore be made against the use of high strength steel tensioned to a multiple of the usual permissible stresses. However, to fulfil the condition of full prestressing a great stretching force is required and the whole required tensile reinforcement has to be stretched. Moreover, the maximum straining of the structure at the release of the precompression on the concrete is greater than that under working load, which necessitates a great compressive strength at the early stage and is inconvenient for the

transport and handling of precast products, since any additional straining has to be avoided.

By contrast, in effective partial prestressing a considerably smaller stretching force is applied with resulting economy, when only a part of the reinforcement is stretched. The reduction of the stretching force itself may not effect a saving in cost, if the stretching process is carried out by a machine, but the reduction of the number of rods to be stretched is of importance since by a simultaneous tensioning each individual rod has to be gripped at its ends and an equal straining in each rod has to be ensured before stretching the rods. Due to the reduction of the stretching force the initial maximum straining is greatly reduced as against full prestressing; however, cracklessness is not guaranteed. In spite of the substantial reduction of the reinforcement, which may be decreased to one-fifth or even less, according to the aim of partial prestressing, a straining is obtained under working load similar to that in a conventionally reinforced structure. The theoretical concrete tensile-bending stresses in a straight stress distribution serve as a basis of comparison. In a cracked section under working load there appear theoretical steel stresses which are a multiple of the usual permissible stresses. However, tests have proved that dangerous cracks which would occur if the whole reinforcement were unstretched, are avoided, as long as the reinforcement or at least a considerable part of it, is bonded to the concrete. The high steel stresses are developed only, where the bond is destroyed, in a short length in the neighborhood of each crack, especially when high strength concrete and thin wires, ensuring a good bond, are used.

A partly prestressed structure represents in its behavior an intermediate case between a fully prestressed homogeneous structure with very small deformations under working load and an unstretched structure in which great deformations and heavy cracking are developed, when high strength steel is used. The design has, of course, to ensure the same factor of safety against breaking both for fully and partly prestressed concrete, as discussed in Chapter 4. A comparison of a fully and partly prestressed design is shown in Table 3 and described more in detail in Chapter 9.

4. BEHAVIOR OF PRESTRESSED REINFORCED CONCRETE

In reinforced concrete, at cracking, a transformation of the behavior takes place, as can be seen from the break in the deformation diagram, there being in principle no difference between conventionally designed and prestressed reinforced concrete. In a cracked section of a prestressed structure, the pre-compression which is produced mainly in the normal tensile zone is gradually reduced or totally interrupted. As long as the cracks do not widen too much and consequently no substantial permanent

elongation of the steel remains, the cracks close at the removal of the load, and the pre-compression is transmitted between the adjacent parts. Thus the stress distribution before cracking is as with a homogeneous material of some plasticity, as concrete, the pre-compression and the counter-bending being taken into account. These counteractions gradually disappear when wide cracks occur and at failure apparently only a conventionally cracked section—without any pre-compression and counter-bending—has to be considered.

Since with the use of high strength steel the percentage of the reinforcement is greatly reduced, there is always a state of under-reinforcement in such prestressed concrete—assuming the latter is of high strength, which is the normal case—and thus only the steel primarily influences failure. "It is agreed" on the basis of long experience in prestressing, according to Freyssinet's system, "that prestressed concrete presented no advantage as regards the point of ultimate failure, no claim has been advanced in that respect."[9]

This has also been proved in tests by Hoyer[8] on four series of beams of the same cross section, reinforced with wire of a strength of 369,000 psi., but differing in initial prestress (14,220, 56,880, 113,760 and 170,640 psi).* The ultimate load was in each case nearly the same. The two beams of each series were investigated simultaneously. In one of the beams with the highest prestress the wire broke; the other beam, however, remained intact and yielded laterally apparently effecting an unequal load distribution. At failure, five of the other beams crushed in the compression zone, the sixth beam remaining intact by laterally yielding. In all beams the theoretical steel stress at failure, calculated for a cracked section in simple bending, was 330,000 psi. i.e. 89.4 per cent of the ultimate strength of the wire. In view of the relatively small difference between the theoretical steel stress and the strength, which is only a small fraction of the initial prestress, and in view of other test results available, conclusions cannot be drawn from the fact that the wire broke only in one of the beams with the highest prestress.

Reference may also be made to tests conducted by Emperger[10] relating to four series of beams, one reinforced with mild steel of a yieldpoint of 38,350 psi. and the others with twisted round bars of special shape (Torstahl) of an equivalent yieldpoint of 57,000 psi., all having an additional bonded wire reinforcement (equivalent yield point 170,400 and ultimate strength 213,000 psi.), these wires, employed in couples, being twisted and threaded with cross bars to increase the bond. In all these series the ultimate loads obtained by tests agree surprisingly well with the theoretical loads calculated for a cracked bending section, as

*See Table 3 in (8) and Tables IV and VII in the writer's contribution to (9).

TABLE 2. THE TESTS OF EMPERGER

Series		1	2	3	4
Shape of beam		Rectangle (1)	T-profile (2)	Rectangle (1)	T-profile (2)
Percentage of reinforcement %		1.07	0.248(3)	0.325	0.113(3)
Main Reinforcement	Mild steel %	95	—	—	—
	Tor steel %	—	81	64	58
Additional Wire %		5	19	36	42
Theoretical steel stress in wire at failure		f'_s	f_y	f_y	f'_s
State of wire at failure		broken	unbroken (4)		broken
Ultimate load / Theoretical maximum load		1.04	0.987	1.01-1.04	0.99

(1) b = 30 cm., D = 25 cm., d = 24 cm.
(2) B = 38 cm., b = 16 cm., D = 33 cm., d = 31 cm., d_s = 11 cm.
(3) related to $B.d$.
(4) In some of the beams series 3, first the main reinforcement and afterwards also the wires broke.

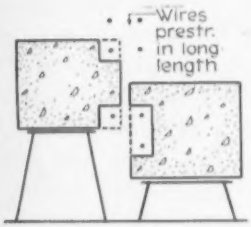
can be seen from Table 2. With series 2 and 3 the additional wire did not break as with series 1 and 4. Emperger says that with some of the beams 3 (6 beams of this series and 2 beams of each other series were tested), the main reinforcement of a relatively small ultimate elongation (Torstahl) broke. Since further details are not disclosed, it is impossible to find out why with some of these beams the main reinforcement broke, although the ultimate load agrees very well with the equivalent yield-point stress. It is also not clear, why the wire broke in series 4 and not in series 2 and 3, although in all three series the same main reinforcement (Torstahl) was used. This matter therefore requires further investigations. It may be added that in all four series the initial prestress was in accordance with Emperger's proposal "moderate low" (57,000 psi.) and low-strength concrete was employed (Cube strength 3200-3550 psi. corresponding to f'_c of 2400 to 2660 psi.).

Since the exact straining at failure in a conventionally designed structure has not yet been known—especially in under-reinforced structures with well bonded high strength steel reinforcement[11]—, a full clarification cannot be expected for prestressed concrete. In view of the above considerations and test results discussed, it can be assumed that the ultimate tensile resistance is not increased by prestressing, and

the same permissible steel stresses, related to ultimate steel stress and ensuring the same factor of safety, have therefore to be taken into account for full and partial prestressing. However, this is quite correct only as far as prestressed concrete with bonded reinforcement is concerned. When there is no bond between the prestressed reinforcement and the concrete, the permanent deformation of the steel is much greater, since it occurs not only in the neighborhood of cracks but along the whole length; consequently the cracks are wider and the structure does not behave like a conventional reinforced concrete beam but like a two hinged arch having a tie[12]. It will depend on further investigations whether a prestressed concrete beam without any bond is able to carry at failure the same loads as an equivalent beam with bonded reinforcement, if high strength steel is used. Of course, all these considerations relate only to a design where failure owing to shear or slip of the reinforcement is prevented. On the whole, with full prestressing the shear resistance is greatly enhanced as described more in detail in[4] [3] and.[13]

5. A NEW METHOD OF PRESTRESSING

The new methods of Schorer[2],[4] and Billner[5] appear especially useful for full prestressing (3*b*) and partial prestressing (6*a*). The superiority of these new methods compared with methods formerly known has been stressed in Chapter (1).



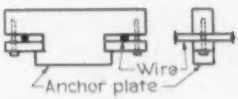


Fig. 1 and 2 (at the right)

A further new method relating to post-stretching, suggested by the writer,* may also be mentioned. The concrete products or structures cast in place have recesses provided along the tensile surface, as shown in Fig. 1. It is thus possible to manufacture them by mass production or in the usual manner in molds. After the concrete has hardened and attained sufficient strength, the stretched reinforcement is placed in these recesses and the pre-compression transmitted to the concrete. The stretching may be carried out direct against the concrete, special anchor plates being arranged at the ends which ensure the correct positioning of the reinforcement and avoid its touching the concrete in the

*Brit. Pat. No. 556,572.

recesses (friction would prevent uniform tensioning). If the reinforcement consists of thin wire, two recesses may be formed at the two edges (see Fig. 1 left part), the wire wound round the whole product, which may also consist of an assembly of single blocks), and the stretching process be carried out by twisting the wires, preferably by machine, the prestress being ascertained by the tone. For constructions cast in place the tensioning may be carried out by winding wires around a reel anchored in the hardened concrete. There is, however, another possibility of simplification in the manufacture of precast products i.e. to stretch the wire in long length against anchor blocks at its ends. The hardened products are brought into position, as shown in Fig. 1. At the ends of each product simple anchor devices may be used, having jaws to grip the tensioned wire according to Fig. 2, relating to Fig. 1 left beam.

It is possible to fill these recesses completely with concrete or cement mortar and thus to obtain an additional bond. This method allows therefore of a combination of the advantages of post-stretching and pre-stretching similar to Billner's method, as pointed out in 1. It is, however, also possible to fill the recesses with a plastic cohesive filler, thus protecting the reinforcement and allowing of a later readjustment of the tensioning, which might be of some advantage for cast in place constructions.

An idea similar to the latter was already suggested by Walter C. Parmley, of Upper Montclair, New Jersey, in 1927* for fully prestressed concrete pipes. According to this suggestion, steel hoops are placed in grooves, provided in such pipes, and are tensioned, the grooves afterwards being filled with a non-adhesive material. However, by post-stretching of the hoops it is impossible to ensure a uniform straining along the circumference in view of the friction between hoop and concrete. With circular reinforcements, however, an accuracy of tensioning is attained by use of a new wire winding machine[14], the wire being tensioned (e.g. to about 150,000 psi.) and wound around concrete shells for prestressed tanks.

6. LOSSES OF THE INITIAL PRESTRESS

The initial prestress p_i† is reduced, as mentioned in part 1, in the case of pre-stretching by the losses owing to elastic deformation Δp_e and to shrinkage Δp_s and in the case of "post-stretching" by the loss, owing to that part of shrinkage Δp_{s2}, which takes place after the pre-compression is transmitted to the hardened concrete ($\Delta p_{s2} = \Delta p_s - \Delta_{s1}$, Δp_{s1} denoting the loss, owing to that part of shrinkage, occurring before transmission).

*U. S. Pat. No. 1,781,699 (Patented 1930).
†Notation see Appendix.

In both cases also the loss, owing to plastic flow, Δp_p has to be considered. Hence the remaining prestress at working load:

Pre-stretching: $p_w = p_i - \Delta p_e - p_s - \Delta p_p$ ..... (1a)

Post-stretching: $p_w = p_i - \Delta p_{s2} - \Delta p_p$ ..... (1b)

On the transfer of the pre-compression to the concrete, only a part of the total loss due to shrinkage Δp_{s1} has to be taken into account in the case of pre-stretching and no loss occurs with post-stretching. Hence the remaining pre-stress p_t at the transmission of the compression to the concrete:

Pre-stretching: $p_t = p_i - \Delta p_e - \Delta p'_{s1}$ ..... (2a)

Post-stretching: $p_t = p_i$ ..... (2b)

When only the bottom reinforcement is prestressed the loss due to the elastic deformation of the concrete in a section according to Fig. 3 amounts to:

$$\Delta p_e = n\left(\frac{P_i}{A_o} + \frac{P_i.e^2_{so}}{I_o}\right) = \frac{n.P_i}{A_o}\left(1 + \frac{e^2_{so}}{g^2}\right) \quad \ldots\ldots (3)$$

For a rectangular section (breadth b, total depth D, prestressed bottom reinforcement, as conventional, $A_s = p.b.d.$), equation (3) can be simplified if the influence of the top reinforcement A'_s is neglected $\left(g^2 = \frac{1}{12} D^2\right)$:

$$\Delta p_e = n.p.p_i.\frac{d}{D}\left[1 + 3.\left(\frac{2d}{D} - 1\right)^2\right] \quad \ldots\ldots (3a)$$

From this equation it is seen that the loss is reduced for the same p_i with a decreasing percentage of the reinforcement p. The relative loss of prestress becomes therefore a minimum with a maximum initial prestress and a minimum tensioned reinforcement. For $n = 8$ and $\frac{D}{d} = 1.1$, the equation (3a) can be transformed to

$$\frac{\Delta p_e}{p_i} = 17.5p \quad \ldots\ldots (3b)$$

p being expressed in per cent. For a reinforcement of 0.1 per cent the loss of the initial prestress is only 1.75 per cent and increases to 17.5 per cent for a reinforcement of 1 per cent.

When the top reinforcement is also prestressed according to Fig. 4, P'_i and p_i being the initial stretching force and prestress at the top, the two equations (4a) and (4b) are derived.

$$\Delta p_e = n.\left[\frac{P_i + P'_i}{A_o} + \frac{(P_i.e_{so} - P_i.e'_{so}).e_{so}}{I_o}\right] \quad \ldots\ldots (4a)$$

$$\Delta p'_s = n.\left[\frac{P_i + P_i}{A_o} - \frac{(P_i.e_{so} - P'_i.e'_{so}).e'_{so}}{I_o}\right] \quad \text{(4b)}$$

The shrinkage coefficient varies between 0.00025 and 0.0005. The loss owing to plastic flow depends on the magnitude of the concrete stresses developed and on their duration. The maximum may be considreed to be not greater than the greatest loss due to shrinkage. To be on the safe side it suffices to consider the greatest possible losses only, which amount on the above simplified assumption to $0.001E_s$ i.e. to 25,000 to 30,000 psi. for a Modulus of Elasticity of 25 to 30 x 10^6 for high strength steel.

From all these considerations it is seen that for a larger reinforcement and a smaller initial prestress a much greater initial stretching force P_i has to be applied than for a smaller reinforcement and a higher initial prestress to ensure the same stretching force P_w remaining at working load conditions.

7. THE CONDITIONS FOR FULL PRESTRESSING

a. Bottom reinforcement only prestressed (Fig. 3)

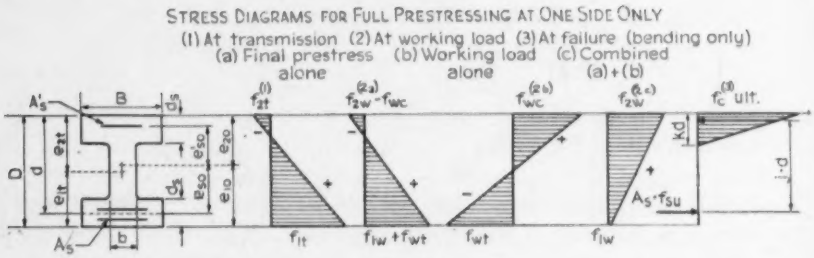


Fig. 3

The main equation for the design of a fully prestressed section can be derived from the condition that the stress f_{1w} must not be a tensile stress, the limit being $f_{1w} = 0$.

The stresses at working load are given in the following equations (5a) and (5b), M_w denoting the bending moment for working load, and the area A_o and the moments of inertia I_o and I_t computed as follows:

$$A_o = b.D. + 2.(B\text{-}b).d_s - A_s + (n\text{-}1).A'_s, \text{ and } A_t = A_o + n.A_s$$

$$I_o = \frac{1}{12}\left[B.D^3 - (B\text{-}b).(D - 2.d_s)^3\right] - A_s.e^2_{so} + (n\text{-}1).A'_s.e'^2_{so}, \text{ and}$$

$I_t = I_o + A_o.(e_{1o} - e_{1t})^2$. $A_o, I_o, e_{so}, e'_{se}, e_{1o}$ and e_{2o} relate to the net area of the concrete and to the unstretched area, whereas A_t, I_t, e_{1t} and e_{2t} relate to the total area including the prestressed reinforcement, there being the following relation: $n.A_s.e_{so} = A_t.(e_{1o} - e_{1t})$.

$$f_{1w} = \frac{P_w}{A_o} + \frac{P_w.e_{so}.e_{1o}}{I_o} - \frac{M_w.e_{1t}}{I_t} \quad \ldots\ldots (5a)$$

$$f_{2w} = \frac{P_w}{A_o} - \frac{P_w.e_{so}.e_{2o}}{I_o} + \frac{M_w.e_{2t}}{I_t} \quad \ldots\ldots (5b)$$

Equation (5a) can be transformed for the limit $f_{1w} = 0$ to

$$p_w = \frac{P_w}{A_s} = \frac{M_w}{\left(\frac{I_t}{A_o.e_{1t}} + e_{so}.\frac{I_t.e_{1o}}{I_o.e_{1t}}\right).A_s} = \frac{M_w}{\left(\frac{S_{1t}}{A_o} + e_{so}.\frac{S_{1t}}{S_{1o}}\right).A_s} \quad \ldots (6)$$

hence the initial stretching force, considering the losses Δp according to equation (1a) or (1b)

$$P_i = A_s.p_i = A_s.(p_w + \Delta p) \quad \ldots\ldots (7)$$

Equations (6) and (7) are the main conditions for the determination of the required minimum stretching force to ensure full prestressing. When the percentage of reinforcement is small, the center of gravity of the unreinforced concrete section A_c will only be slightly shifted for the sections A_o and A_t, since n is rather small (about 8) in this stage of homogeneity, e_{1o} and e_{1t} may therefore be considered to be approximately equal.

The reinforcement A_s has, of course, to ensure the desired factor of safety s against breaking, as pointed out in Chapter 4, in accordance with equation (8), relating to a cracked section of conventional type, f_{su} being the ultimate steel stress at failure.

$$A_s = \frac{M_w.^s}{j.d.f_{su}} \quad \ldots\ldots (8)$$

For high strength steel, especially wire, f_{su} may be 85 to 100 per cent of the total strength and mostly exceeds the so called proof stress (corresponding to a permanent elongation of 1 per cent).

The greatest stresses do not occur at working load but at the transmission of the pre-compression to the concrete, as is seen in Fig. 3, in accordance with the following equations (9):

$$f_{1t} = \frac{P_t}{A_o} + \frac{P_t.e_{so}.e_{1o}}{I_o} = \frac{P_t}{A_o} + \frac{P_t.e_{so}}{S_{1o}} \quad \ldots\ldots (9a)$$

$$f_{2t} = \frac{P_t}{A_o} - \frac{P_t.e_{so}.e_{2o}}{I_o} = \frac{P_t}{A_o} - \frac{P_t.e_{so}}{S_{2o}} \quad \ldots\ldots (9b)$$

These stresses are reduced at the center portions of beams when the dead weight counteracts the straining due to the prestress. The expression $P_t.e_{so}$ in the above equation has for this case to be replaced by $(P_t.e_{so} -$

M_o), M_o denoting the bending moment at the particular sections for which the stresses are calculated due to dead weight in action before the transmission of the pre-compression of the concrete.

b. Top and bottom reinforcement prestressed

The occurrence of tensile stresses f_{2t} (Fig. 3) can be avoided, if the top reinforcement also is prestressed, as illustrated in Fig. 4. In this case the two equations (5a) and (9b) serve together with the equations (4a) and (4b) for the determination of the two forces P and P'.
corresponding to equation (5a):

$$f_{1w} = \frac{P_w + P'_w}{A_o} + \frac{P_w.e_{so} - P'_w.e'_{so}}{S_{1o}} - \frac{M_w}{S_{1t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

corresponding to equ. (9b):

$$f_{2t} = \frac{P_t + P'_t}{A_o} - \frac{P_t.e_{so} - P'_t.e'_{so}}{I_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

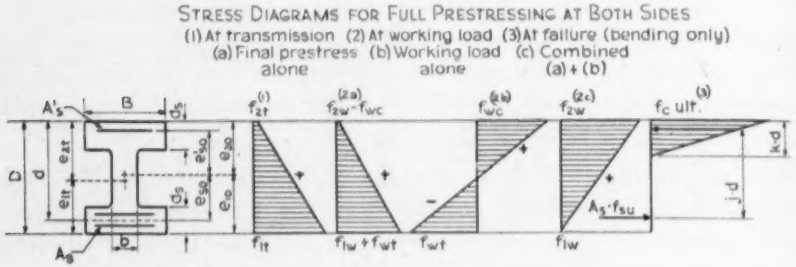


Fig. 4

These equations have to be transformed for the limits $f_{1w} = 0$ and $f_{2t} = 0$.

8. SAFETY AGAINST CRACKING

The factor of safety against first cracking* s_r is given by the equation:

$$s_r = \frac{f_{1w} + f_{wt} + f_r}{f_{wt}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

f_{wt} is the tensile stress, due to working load, in a homogeneous section with unstretched reinforcement. The expression $(f_{1w} + f_{wt})$ represents the remaining prestress at the tensile fibre after the greatest possible losses have taken place (Fig. 3 and 4). In a fully prestressed structure, f_{1w} is a compressive stress with the limit zero. The stress f_r, occurring at first cracking (rupture), corresponds to the flexural strength f'_{cf} of unreinforced concrete, which is also called "Modulus of rupture." This is only

*Under "cracking" it is understood in the following that state where the first cracks become visible.

an arbitrary value, obtained for a straight line stress distribution and incapable of measurement. But in practice, the stress distribution in the tensile zone of a reinforced concrete section before cracking, corresponding to failure in an unreinforced section, is either curved or rectangular, due to the plasticity of the material, the greatest stress being equal to the tensile strength of the concrete f'_{ct} (see [4] page 528-5 Fig. B and C). The Modulus of rupture varies greatly for different shapes and dimensions of the cross section and for different plasticities, even if the tensile strength is the same, and amounts to about 1.5 times to twice the tensile strength. The latter is, however, "not appreciably affected by either length or size" of specimens, as tests have proved.[15] The concrete tensile strength can be obtained from briquettes or, preferably, cylinders. It is known that the stress distribution in briquettes is not uniform, owing to the reduction of the cross section[16]; however, some tests, discussed in the following, have proved quite satisfactory. Several formulas are known for the relation between f'_{ct} and f'_c, mainly on the basis.....

$f'_{ct} = a.f'^{b}_{c}$, *e.g.* $f'_{ct} = 5\sqrt{f'_c}$ according to Abrams ([4] formula (1)). If generally $f'_{ct} = \frac{f'_c}{k}$, the value k is according to new extensive investigagations [17] for a (1:4.5) mix concrete on the average 8 for $f'_c = 2000$ psi. and 15 for $f'_c = 8000$ psi. Thus the following relation is obtained for intermediate values $k = 8 + \frac{f'_c - 2000}{857}$, which gives a good approximation.

When f'_{ct} is known, f_r can be replaced by 1.5 times to twice f'_{ct}. It appears to be advisable, generally, to consider only the smaller value *i. e.* $1.5\,f'_{ct}$ in order to include all cases of plasticities and various shapes and dimensions.

When considering the factor of safety against cracking, necessary to guarantee complete absence of cracks, the effect of repeated loading has to be taken into account. According to various tests, cracking in a reinforced concrete structure of conventional design can only be avoided, if the working load is less than half the load which results in first cracking. [18] It means that the concrete tensile stress f_{rr} at repeated loading, and still avoiding cracking, must not be greater than $\frac{1}{2} f_r$ or $0.75\,f'_{ct}$. With repeated loading, however, only the live load and not the working load has to be considered. The relation $f_{rr} = 0.75\,f'_{ct}$ is therefore only correct when the dead weight is negligibly small compared with the total working load. Assuming the dead weight is half the working load,

the reduction of the stress f_r at first cracking is not $\frac{1}{2}$ but $\frac{1}{1.5}$, hence $f_{rr} = f'_{ct}$, which may be considered as a general relation, giving the equation:

$$s_{rr} = \frac{f_{1w} + f_{wt} + f'_{ct}}{f_{wt}} \quad \ldots\ldots (12a)$$

When the live load is ¾ or ¼ of the total working load, the equation $f_{rr} = f'_{ct}$ has to be replaced by $f_{rr} = \frac{1.5}{1.75} f'_{ct} = 0.86 f'_{ct}$ or $f_{rr} = \frac{1.5}{1.25} f'_{ct} = 1.2 f'_{ct}$ respectively. Since with prestressing the dead weight is greatly reduced, it is more likely that the live load is at least ½ of the total working load with the exception of roofs, where the dead weight is of main influence. When the live load is more than half the total working load, f'_{ct} in equation (12a) should be replaced by a smaller value, say $0.8 f'_{ct}$. (limit $0.75 f'_{ct}$).

9. PARTIAL PRESTRESSING

For the design of this kind, equation (8) remains unaltered for the reinforcement $A_s = A_{sp} + A_{su}$, if the same quality of steel is used for each part. If the properties of the prestressed and unstretched reinforcement are different, equation (8) has to be replaced by (8a), in accordance with the tests of Emperger[10], f_y denoting the yield point stress or its equivalent for the unstretched reinforcement:

$$A_{sp} + \frac{f_y}{f_{su}} A_{su} = \frac{M_w.s}{j.d.f_{su}} \quad \ldots\ldots (8a)$$

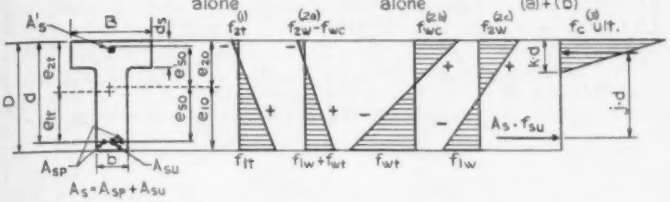
Fig. 5 shows the stress distribution for a partly pre-stressed T-section. The stresses occurring at the release of the stretching force are much below those with full prestressing and do not require a specially increased compression portion at the normal tensile zone (which is the case in an I-section).*

The resultant tensile stress f_{1w} may serve as a basis of comparison with conventional practice. It is not usual to compute this stress, since the design is carried out for a cracked section only. In order to have some idea of the magnitude of straining in conventional structures, designed for a permissible steel stress $f_s = 18{,}000$ psi., the concrete tensile stress f_{wt} at working load for a homogeneous section ($n = 8$) is shown in Fig. 6 (both for a rectangle $\frac{D}{d} = 1.1$ and a thin slab of $\frac{D}{d} = 1.4$) in relation

*In Fig. 5 a reduction of the stresses due to the pre-compression is indicated. Such a reduction would occur, if the cross section were the same as in Fig 3 and 4. However for a T-section according to Fig. 5 the compressive stress at transmission f_{it} would, in view of the smaller concrete portion, not be reduceed as compared with the equivalent stress for full prestressing in an I-section.

STRESS DIAGRAMS FOR PARTLY PRESTRESSING **Fig. 5**

(1) At transmission (2) At working load (3) At failure (bending only)
(a) Final prestressing alone (b) Working load alone (c) Combined (a) + (b)



to the percentage of the reinforcement p and to the compressive stress f_c (in a cracked section, n = 15). The stresses f_{wt} vary between 51.5 psi. for a thin slab of 0.1 per cent reinforcement and 1080 psi. for a rectangle (designed for the limit f_c = 1250 psi), which latter tensile stress increases when a compressive reinforcement is provided. Even higher stresses are obtained for T-beams in which case the centre of gravity is nearer to the upper fibre and the stress f_{wt} may increase to 1500 psi. and even more for structures designed in accordance with the existing regulations.

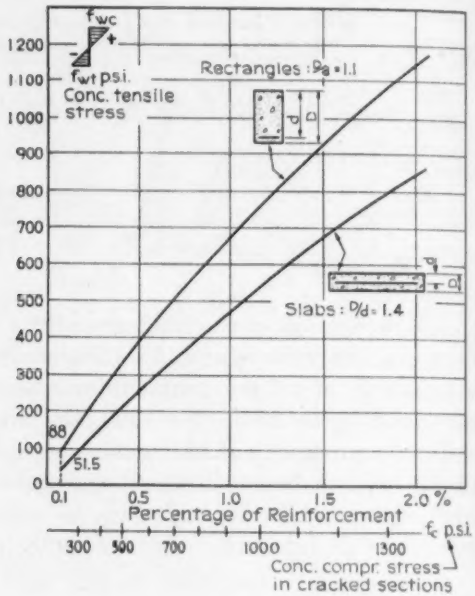
It is necessary to determine the limit of the stress f_{1w} for guaranteed absence of cracks. As can be seen from the foregoing chapter, total absence of cracks is ensured when the tensile stresses f_{1w} are considerably less than the stress f_r corresponding to first cracking. From equation (12a) it is seen that cracklessness cannot generally be guaranteed when the stress f_{1w} is greater than the concrete tensile strength f'_{ct} which limit has to be reduced to 0.75 f'_{ct} for an increased ratio of live load to total working load.

The reason why dangerous cracks do not develop in partly prestressed structures-although the theoretical steel stress in a cracked section is 100,000 psi. and more—has been discussed generally in Chapter 3. In a structure in which a part of the tensile reinforcement is unstretched, the pre-compression of the untensioned reinforcement is also of favorable influence.*

A reduction of the stretching force, derived in equation (6) for full prestressing, to about 40 per cent has proved sufficient in a special instance, as will be seen in Chapter 11. Further tests are required to prove that, generally, ¼ to ½ of the stretching force for full prestressing suffices to avoid dangerous cracks. In Table 3, three beams of the same cross section and reinforcement (Fig. 7) are compared for (a) unstretched, (b)

* Mr. Schorer states in his closure to (4), page 528-7 that "the unstretched reinforcement will in time be subjected to considerable compressive stresses which require closely spaced stirrups, lateral ties or spirals, as in columns, in order to counteract possible buckling tendencies of the longitudinal bars." This is only correct for a reinforcement in the compressive zone of a beam, where the straining is increased with increasing load. But the unstretched reinforcement in the tensile zone of a prestressed beam is pre-compressed only to a relatively small extent, say to 10,000—15,000 psi. At the release of the stretching force this stress remains only at the ends of the reinforcement and is transformed into a tensile stress in the greatest part of the beam, the maximum tensile stress, in a cracked section, being a multiple of the original compressive stress. No special provisions againt buckling are therefore necessary.

Fig. 6



partly prestressed and (c) fully prestressed wires. It is interesting to note that the stress f_{1w} for the partly prestressed structure, reinforced with high strength wire, is about the same as the stress f_{wt} computed for a beam of the same carrying capacity but reinforced with mild steel.

10. DISCUSSION OF SOME PUBLISHED COMPARATIVE TESTS

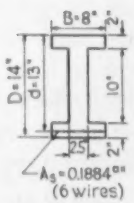
Comparative tests on prestressed beams with post-stretched reinforcement of mild steel and conventionally reinforced beams have been carried out by W. K. Hatt [19] and R. E. Mills and W. B. Miller.[20] The first relate to 4 beams 8 in. by 14 in., 13 ft. 6 in. long of 12 ft. span, reinforced with 2 rods ($p = 1.04$ per cent), 3 beams prestressed to different extents and one conventional beam, whereas the second tests were carried out on 3 I-shaped joists 3 in. by 8 in., 12 ft. long of 11 ft. span, reinforced with one rod of ¾ in. diam. ($p = 2.2$ per cent), 2 joists of Haydite concrete (one prestressed and one conventional) and one joist of segmental synthetic stone. The test results do not allow of comparison of the behavior up to the ultimate load, since the first tests relate only to stresses in the conventionally reinforced beam up to 26,000 psi., whereas in the second group of tests the joists with bonded unstretched reinforcement failed in shear (at about 2/3 of the ultimate load attained with the prestressed joist of the same material). Both groups of tests have shown that, as long as no cracks develop, the behavior of the prestressed article agrees very well with that expected according to calculation for a homogeneous

TABLE 3—COMPARISON OF 3 I-SHAPED BEAMS (SEE FIG. 7) OF THE SAME CARRYING CAPACITY

Designed for high strength steel reinforcement: $f_{su} = 200{,}000$ psi., $z = 2/3\,d = 12$ in.; $M_{ult} = 0.1884 \times 12 \times 200{,}000 = 452{,}160$ lb. in. hence $M_{perm} = 226{,}000$ lb. in. corresponding to a uniformly distributed load of 377 lb. per ft. for a simply supported beam of 20 ft. span (dead weight: 57 lb. per ft.), greatest shear stress: 126 psi. (sufficient shear reinforcement provided). — Calculated with the approximate design values: $A_c = A_t = A_c = 57$ in.^{2}, $e_{1o} = e_{2o} = e_{1t} = e_{2t} = \frac{D}{2} = 7$ in., $e_{ss} = 6$ in. and $S_{1o} = S_{2o} = S_{1t} = S_{2t} = 196$ in.^{3} (No. 8).

			(a) Reinf. unstretched	(b)+ Partly prestr. (⅔)	(c) Fully prestressed
P_i for p_i = 170,000 p.s.i.	lb.		–	10,667	32,000
Δp_e according to equ. (3)	p.s.i.		–	3,733	11,200
$p_t = p_i - \Delta p_e - \Delta p_{sl}$ ($\Delta p_{sl} \sim 6300$ p.s.i.)			–	160,000	152,500
P_t	lb.			10,000	28,700
Stress distribution at transmission of the pre-compression	p.s.i		–	$f_{2t} = -130$, $f_{1t} = +482$	$f_{2t} = -376$, $f_{1t} = +1386$
Deformation of the beams at transmission of the pre-compr. (before loading)					
$p_w = p_t - 24{,}000$ (assumed $\Delta p_{ss} + \Delta p_p \sim 30{,}000 - \Delta p_{sl}$)	p.s.i.		–	136,000	128,500
P_w	lb		–	8,540	24,200
Concrete stresses in a straight line stress distribution under working loads	p.s.i.		$f_{wc} = 1155$, $f_{wt} = 1155$	$f_{2w} = +1044$, $f_{1w} = -743$	$f_{2w} = +838$, $f_{1w} = +13$
Deformations of the beams when under working load					
Safety against cracking	s_r++ (equ. 12)	$f'_{cf} = 750$ p.s.i.	0.65	1.07	1.66
		$f'_{cf} = 400$ p.s.i.	0.35	0.70	1.36
	s_{rr}++ (equ. 12a)	$f'_{ct} = 500$ p.s.i.	0.43	0.79	1.44
		$f'_{ct} = 250$ p.s.i.	0.22	0.57	1.23

Fig. 7



NOTES: +The steel consumption of the tensile reinforcement for the partly prestressed beam can be reduced by 19% as compared with the fully prestressed beam, which needs, say, 1.05 A_s (5% for overlength) as against $1.05 \frac{A_s}{3} + \frac{2.3}{3.4} A_s = 0.85\, A_s$ (assuming the unstretched reinforcement (2/3, A_s) could be reduced on the average, to ¾ of the length in accordance with the moment distribution).

*Heavy cracking must be expected considering the high tensile stress f_{ct} and the small percentage ($p = 0.18\%$). A conventionally reinforced beam would require a mild steel reinforcement $A_s = 1.04$ in. ($f_s = 18{,}000$ psi.), in which case stresses $f_{uc} = 1078$ and $f_{ut} = 873$ psi. in a straight line stress distribution would be obtained i. e. similar to the stresses f_{2w} and f_{1w} of beam (*b*).

++It is assumed $f'_{cf} = 750$ and $f'_{cf} = 400$ to correspond to $f'_{ct} = 500$ and 250 psi respectively. For beam (*a*) the effective prestress $f_{1w} + f_{wt} = 0$.

material. From Chart 7 in the publication [20], however, it is seen that in the Haydite joist with destroyed bond, after a crack has developed, the deflection increased to a much greater extent than in the conventional joist.

The tests of Hoyer [8] and Emperger [10], with respect to the behavior at failure, have already been discussed in Chapter (4). As to the behavior before failure they agree very well with the investigations of R. H. Evans [12], [13], discussed more closely in the following. Beams 4 in. by 10 in. of 11 ft. length (10 ft. span) were alternately reinforced with high tensile steel (one rod 1 in. diam. at the bottom in the tensile zone, p = 2.4 per cent and one rod 5/8 in. diam. at the top) and with high tensile wire (tensile reinforcement of 44 wires: A_e = 0.221 in.^{2} at the bottom, p = 0.68 per cent, and 13 wires: A'_e = 0.0654 in.^{2} at the top); 76 per cent of the stretching force was applied to the bottom reinforcement and 24 per cent to the top reinforcement in order to avoid tensile stresses at the release of the stretching force to the concrete.

Comparative tests were carried out for rod and wire reinforcement for the following initial stretching forces: 28,000, 33,600, 47,000 and 56,000 lb. as well as for unstretched reinforcement. A further test was made for a rod reinforcement with a stretching force of 56,000 lb. but destroyed bond. Fig. 8 shows the results on rod beams (including a tests with a initial stretching force of 40,000 lb.) on a chart relating to the greatest central deflection for various loadings, wherein are plotted also the theoretical values for a homogeneous section (n = 8.75) and for a cracked section. It is very striking that the deflections of the prestressed beam with destroyed bond are much greater than both those in the bonded type and the theoretical values for the conventional type, even before cracks have developed. Evans has derived formulas for the steel and concrete stresses in such beams on the assumption that the total strain in the ties must equal the total strain in the concrete surrounding it [12]. The values thus obtained agree very well with the measurements. When cracks have developed, reinforced concrete beams with destroyed bond behave as two-hinged arches, the deflection immediately after cracking greatly increasing.

As far as the difference between rod and wire reinforcements is concerned, the losses of the initial prestress were in accordance with the equation (3) much greater for the rod beams than for the wire beams, and relatively greater for smaller initial stretching forces than for greater. Cracking thus occurred much earlier in the rod beams than in the wire beams. Fig. 9 shows a comparison of the behavior of rod and wire beams similar to Fig. 8, the original charts for the respective beams being brought to the same basis of comparison by taking into account the different steel areas in the moments of inertia. From Fig. 9 it is seen that it is possible, between the two limits of beams with unstretched reinforcement and fully prestressed beams (stretching force of 56,000 lb. for wire beams), to find any convenient intermediate case where the deformation and cracking is limited to a definite extent. This can be done either by tensioning

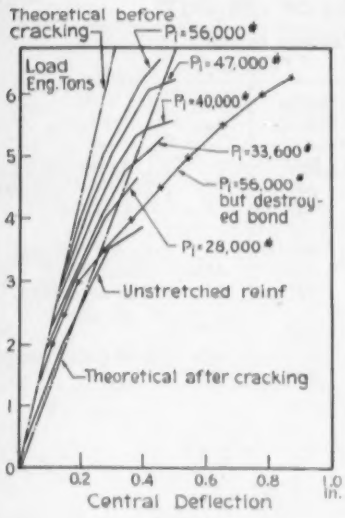


Fig. 8

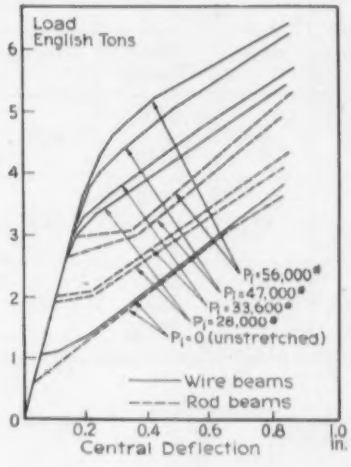


Fig. 9

the whole reinforcement to a lesser degree than necessary for full prestressing, as done in the tests by Evans, or by tensioning a part of the reinforcement only, but to a high initial prestress, a part remaining unstretched and being pre-compressed at the release of the stretching force on the concrete; the latter case is more advantageous.

11. NEW COMPARATIVE TESTS

Preliminary tests on beams reinforced with high tensile wire were made at R. H. Harry Stanger's Testing Institute, London, and conducted by the writer in collaboration with K. Hajnal-Konyi.* It was intended to compare the behavior of three different types of beams, having the same section, being of the same concrete strength, and reinforced with the same amount of high tensile wire, which is (a) not tensioned, (b) to a part (40 per cent) prestressed, the stretching force being 40 per cent of that required for full prestressing, the rest remaining unstretched, and (c) fully prestressed. According to the program two beams for each type were to be tested and two beams were to be manufactured simultaneously. The cross section of the beams and a view, showing the arrangement of loading, are shown in Fig. 10 and 11 respectively. The stress-strain diagram of the wire is shown in Fig. 12. Table 4 contains mix and strength properties of 3 series of concrete.

Unfortunately, it was not possible to use the beams of series 1 in accordance with the program, since the initial film of grease left on the

*Chartered Struct. Eng., London, M.A.C.I.

TABLE 4. STRENGTH OF CONCRETE

Series	Age days	Cube*strength psi.		Tensile strength † psi.		Notes
		Single Values	Average	Single Values	Average	
1	6	5400 5140	5270	469 448 436	451	Concrete Mix: 112 lb. Portland Cement.
2	10	6670 6240	6455	480 467 427	458	1 cu. ft. Stone Court Sand 2.5 cu. ft. Ham River shingle
3	11	5910 5800	5855	382 356 347	362	0.34 water-cement ratio

*cubes 6 in. † briquettes, section 1.5 in. by 1.5 in.

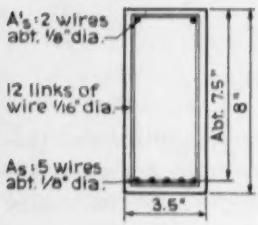
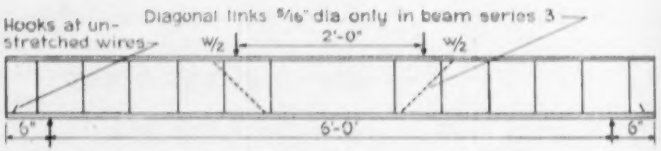


Fig. 10

Fig. 11



wire as protection against corrosion was apparently not wholly removed, by wiping with a clean rag, thus causing an early failure due to slip, owing to insufficient bond, in beams (a) and (c). To avoid bond failure the wires in series 2 and 3 were subjected to a pickling treatment, consisting of degreasing by double treatment with trichlorethylene and etching by immersion in 10 per cent nitric acid for 10 minutes. This treatment proved successful, however, it resulted in a reduction of about 0.004 in. of the diameter of the wires. In series 2 and 3 the time of hardening was extended from 6 days, as in series 1, to 10 and 11 days respectively. Owing to difficulties due to war conditions another brand of cement had to be used for concrete series 3. It was therefore not possible to comply with the main condition of comparing beams of the same concrete strength. As can be seen from Table 4, the strength properties of concrete series 2 and 3 were quite different, the tensile strength of concrete series 3 being greatly reduced as compared with series 2.

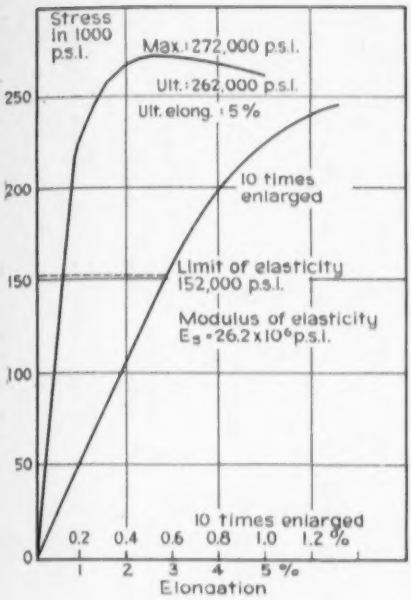


Fig. 12

Test Procedure—Method of prestressing

The wires were tensioned by means of a jack and a cross head to which the single wires were anchored, in such a manner that the stress in each individual wire could be varied by means of screw-thread. First a calibrated jack was used, giving the requisite force by a direct load reading, the single wires being adjusted so that by tuning each wire appeared equally stressed. After the elongation of the wires was recorded, the load was released and the calibrated jack replaced by a positive screw jack, the load applied being sufficient to stretch the wires to the same extent as previously recorded. The reinforcement was kept stretched up to about two hours prior to testing the beams, when the force was released and the contraction of the wires was measured. From this the loss of the initial prestress was ascertained in the case of beam (*b*) of series 3. The data regarding prestressing for beams (*b*) and (*c*) are seen in Table 6.

Method of testing

The deflection was measured at mid-span with a dial micrometer, reading in thousandths of an inch. The formation of cracks was observed and the maximum widths and depths of the cracks were measured under various loads.

Results of the tests

The dimensions of the 4 specimens and the designing values computed therewith are contained in Table 5, whereas in Table 7 there are seen the

TABLE 5—DIMENSIONS OF SPECIMENS AND DESIGN DATA

Concr. ser.	Type	b	D	d	A_s*	p	k.d.**	j.d.**	A_o	e_{so}	e_{to}	e_{tt}	S_{to}	S_{so}	S_{tt}	S_{st}
		in.			sq. in.	%	in.		sq. in.	in.			cu. in.			
2	a	3.50	8.10	7.64	0.0605	0.226	1.74	7.06	.....	.....	.....	4.02	.....	.....	40.5	39.8
	c	3.47	8.10	7.60	0.0609	0.229	1.75	7.02	28.38	3.57	4.07	4.02	38.3	38.6	40.0	39.5
3	a	3.42	8.05	7.72	0.0613	0.232	1.78	7.13	.....	.....	.....	3.99	.....	.....	39.2	38.5
	b	3.45	8.02	7.52	0.0603†	0.233	1.74	6.94	28.13	3.50	4.00	3.98	38.4	38.2	39.2	39.6

* A'_s = 0.0258 sq. in. for all specimens
† A_{sp} = 0.02388 sq. in. (p = 0.09%)

** n = 15
(homogeneous section: n = 8)

TABLE 6—PRESTRESS OF TYPES (B) AND (C)

Concr. ser.	Type	P_i	p_i	P_t	p_t	Total Losses of p_i		p_e (equ. 3)	$\frac{\Delta p_e}{p_i}$	f_{tt}	f_{st}	Notes
		lb.	psi.	lb.	psi.		%	psi.	%	psi.		
2	c	11,300	185,500	9,300	152,500	33,000*	17.8	11,650	7.1	+1180	−556	*Calculated
3	b	4,500	188,500	4,250	178,000	10,500†	5.6	4,100	2.2	+539	−239	†Measured

Note The stress Δp_{st} = 33,000 − 11,600 = 21,350 psi. for beam 2c is too high (with beam 3b only 10,500 − 4,100 = 6,400 psi.), and may be due to insufficient anchorage, which was avoided in the case of beam series 3, type b.

TABLE 7—BENDING MOMENTS AND STRESSES

Concr. ser.	Type	First Cracking					Failure			M_{crack}	Notes
		M	f_1	f_2	f_c	f_s	M	f_c	f_s		
			homogeneous		cracked section			cracked section		M_{fail}	
		lb. in.	psi.				lb. in.	psi.		%	
2	a	32,400	−799	+814	1500	76,000	92,200	4,270	216,000	35.1	
	c	79,200	−800	+1483	3690	185,400	107,150	5,000	252,000	73.8	
3	a	28,400	−725	+738	1300	65,000	73,600	3,380	168,500	38.6	Failure due to shear
	b	53,700	−830	+1153	2565	128,200	85,700	4,100	204,800	62.6	

bending moments and respective stresses at cracking (computed both for a homogeneous section, taking into account the precompression, and for a cracked section in bending only) and the bending moment and respective stresses at failure (computed for a cracked section in bending only).

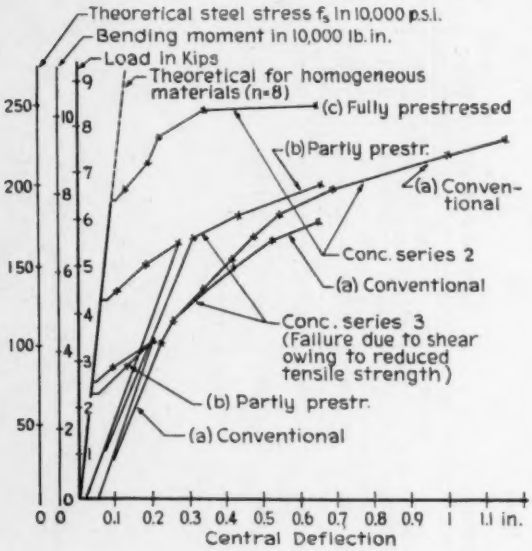

Theoretical steel stress f_s in 10,000 p.s.i.
Bending moment in 10,000 lb. in.
Load in Kips
Theoretical for homogeneous materials (n=8)
(c) Fully prestressed
(b) Partly prestr.
(a) Conventional
Conc. series 2
(a) Conventional
Conc. series 3
(Failure due to shear owing to reduced tensile strength)
(b) Partly prestr.
(a) Conventional
0 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.1 in.
Central Deflection

Fig. 13 (left)

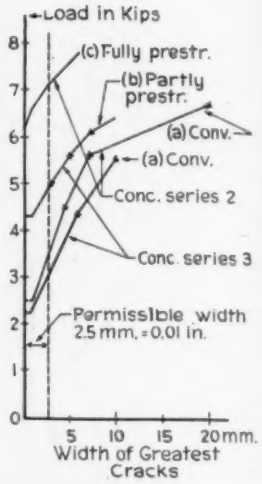


Fig. 14 (above)

Although one of the main bases for a comparison was missing, owing to the difference in the concrete strength, it is possible to compare the results and draw conclusions, since the tests of Evans [12], [13] agree very well, in principle, with the results obtained in series 2 relating to beams a and c and in series 3 relating to beams a and b. Fig. 13 shows the deflections and Fig. 14 the greatest widths of cracks of the beams series 2 and 3 in relation to the loading, bending moment and theoretical steel stress in a cracked ($n = 15$). In Fig. 14 the width of 0.01 in. is also plotted, which is considered to be the limit for dangerous cracks. From the chart it is seen that with beams (*a*) this limit was reached before half the ultimate load, which proves that a wire reinforcement of such high strength cannot be used without prestressing. On the other hand, partial prestressing of 40 per cent was sufficient to delay attainment of such a width until a much higher loading was reached. It is interesting to note that in partly prestressed beams (*b*) the cracks, of a width of 0.02 in. completely disappeared after the load, which was up to 80 per cent of the ultimate load, was removed.

In both beams of series 2 the main crack at which failure occurred was at the loading points, looking as if influenced by shear. For this reason

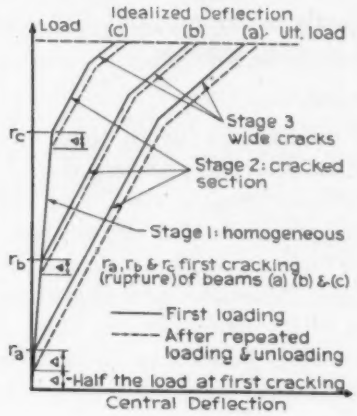


Fig. 15

diagonal closed links were added in series 3 (see Fig. 11). It was expected that this might result in obtaining an even higher ultimate load than in series 2. Since the tensile strength of concrete series 3 was unfortunately much lower than that of 2, the diagonal links prevented only shear cracks at the loading points, but such cracks developed between the latter and the supports, and both beams series 3 failed in shear. The shear stress at failure in beam (*a*) with unstretched wire reinforcement was only 128 psi., which is very low. Further tests are necessary to find the reason.

With reference to the test results shown in Fig. 9 and 13, a comparison of the typical behavior of wire reinforced beams (a), (b) and (c) in idealized form is seen in Fig. 15. Up to cracking the structure behaves as if of homogeneous material ($n = 8$), and after cracking in accordance with conventional practice (cracked section, n about 15), but when the width of the cracks increases, a third stage with greater deformations has to be considered. If sufficient shear resistance is provided for, the ultimate load for all three cases should be about the same, as discussed in Chapter (4).

12. CONCLUSIONS

1. Prestressing generally

The application of this process was of no practical or economical importance until certain improvements were introduced to ensure the effectiveness of the prestress by post-stretching (the losses of the initial prestress, which may exceed with pre-stretching the Limit of elasticity of mild steel, being considerably reduced) and by the use of high tensile steel (the total losses being only a small fraction of the initial high prestress). The ultimate tensile resistance is not increased by prestressing, but both the resistance against cracking and the resilience of the structure

after cracking are greatly enhanced, and the economical employment of high strength steel is made possible.

2. Pre-stretching and post-stretching

There is a distinct difference between these two processes regarding effectiveness of the initial prestress and behavior of the structures. With post-stretching a relatively greater stretching force remains effective than with pre-stretching, thus allowing also conventional steel to be used. However, owing to the lack of bond between reinforcement and concrete, the total and permanent deformations and widths of cracks are greatly increased as compared with constructions in which at least a considerable part of the reinforcement is bonded to the concrete. Therefore when there is no bond, high strength steel can be used only if all possibility of cracking is avoided.

3. Safety against cracking

It is not the load causing the first cracks that has to be considered, but that causing cracking after repeated loading. Total absence of cracks can generally only be attained if the resultant tensile stress at working load in a straight line stress distribution is less than the concrete tensile strength to be obtained from special specimens or related to the cylinder strength. If concrete tensile stresses are excluded altogether ("full" prestressing), there is a definite safety against cracking.

4. Full prestressing

By this process a product is obtained, behaving as if of homogeneous material, hence of particular importance for special constructions such as tanks or barges, where guaranteed cracklessness is essential; an increased shear resistance is attained and steel of the highest strength can economically be used. However, a relatively great stretching force has to be applied, and the whole of the required tensile reinforcement has to be stretched; moreover, the greatest stresses at the release of the precompression on the concrete are greater than those under working load, necessitating a great strength at the early stage, and being inconvenient for the transport and handling of precast products, since any additional straining has to be avoided.

5. Effective partial prestressing

As with full prestressing the use of high strength steel is possible, the disadvantages mentioned above being avoided and the relative loss of the initial prestress becoming a minimum with a minimum tensioned steel and with a maximum initial prestress. Cracklessness is not guaranteed, but dangerous cracks are excluded (although the theoretical steel stresses, appearing in a cracked section, are a multiple of the working stresses

now permitted) because the great elongation of the steel is limited to the immediate neighborhood of the cracks only, where the bond is destroyed, assuming at least a considerable part of the reinforcement is bonded to the concrete. The cracks close totally when the working load is reduced below the cracking load, thus allowing e.g. a construction to be designed which remains crackless under dead load, only fine cracks temporarily occurring under working load. Partly prestressed concrete presents therefore the possibility both of saving steel and of improving the properties. Tests will have to show which part of the minimum stretching force for full prestressing is sufficient to avoid dangerous cracks under working load (presumably ¼ to ½).

6. The performance of prestressing

When whole series are produced a number of wires are tensioned, preferably in long length, against anchorages at the ends. The use of thin wires as stretched and unstretched reinforcement is preferable to that of rods, since it ensures better bond. Except if prestressed reinforcing units according to Schorer [3], [4] are used, with pre-stretching the products have to remain in the mold until the concrete has attained sufficient strength to take up the pre-compression, thus requiring a large working place or a special treatment of the concrete to ensure rapid hardening. This is avoided with post-stretching where there is generally no bond; special mechanical means, however, are necessary for the transmission of the pre-compression to the concrete. There are two methods i.e. that of Billner [5] (electrical prestressing), where the bond is restored later, and the method suggested by the writer, where the stretched reinforcement is placed in recesses of the structure which can later be thoroughly and completely filled with concrete or cement mortar, thus ensuring bond in addition to the direct compression at the ends (the advantages of pre-stretching and post-stretching are thereby combined).

7. Economy of prestressing

Saving of 80 per cent of the tensile reinforcement, required in conventional practice is of some importance when steel is in demand for many other purposes. This saving in cost of material may be 60 to 70 per cent of that of mild steel, which is of great economical importance—at least in Europe. Where labor is of greater influence than the saving of material—as in the United States—the economy of prestressing depends greatly on the method used. The economical superiority of partial prestressing compared with full prestressing can be recognized, when considering the use of prestressed Schorer-units; the amount of this high quality steel is reduced to, say, ½, the other ½ being replaced by unstretched high tensile wire, which is obviously less expensive than the preparation of self-contained units.

8. Prestressed composite structures

It is possible to apply prestressing also to composite structures consisting of an assembly of single blocks, formed from any plastic substance, such as concrete, ceramic, glass, or resinous plastics, and either to thread the reinforcement through perforations or to place it in receses or grooves. In these cases post-stretching is applied and an additional bond may be ensured when the method suggested by the writer is employed.

NOTATIONS

1. General notations

A_s = area of the total tensile reinforcement.
A_{sp} = area of the prestressed tensile reinforcement.
A_{su} = area of the unstretched tensile reinforcement.
A'_s = area of the compressive reinforcement.
E_c = Modulus of elasticity of concrete.
E_s = Modulus of elasticity of reinforcing steel.
f'_c = cylinder strength of concrete, psi.
f'_{cf} = flexural strength of concrete (modulus of rupture), psi.
f'_{ct} = tensile strength of concrete, psi.
f'_s = strength of reinforcement, psi.
f_{su} = theoretical ultimate steel stress (in a cracked section), psi.
f_y = yield point stress or its equivalent, psi.
$n = \frac{E_s}{E_c}$, Modular ratio.
p = percentage of the tensile reinforcement, related to the depth d ($A_s = p.b.d.$).
p' = percentage of the compressive reinforcement.
P_i = initial prestressing force with respect to A_s or A_{sp}, *)
P_t = prestressing force at transmission, *)
P_w = effective prestressing force at working load conditions. *)
p_i = initial prestress in A_s or A_{sp}, psi. *)
p_t = prestress at transmission in A_s or A_{sp}, psi. *)
p_w = effective prestress at working load in A_s or A_{sp}, psi. *)
Δp_e = loss of the initial prestress p_i, owing to elastic deformation of the concrete, psi.
Δp_s = loss of the initial prestress p_i, owing to shrinkage, psi.
Δp_p = loss of the initial prestress p_i, owing to plastic flow, psi.
s = factor of safety against breaking.
s_r = factor of safety against first cracking (rupture).
s_{rr} = factor of safety against cracking after repeated loading.

*) P'_x and p'_x relate to the respective forces and stresses with respect to the reinforcement A'_s (x relating to i, t, w respectively).

2. Cracked section

f_c = concrete compressive stress, psi.
f_s = steel stress, psi.
I = moment of inertia.
$j.d$ = lever arm.
$k.d$ = distance of the neutral axis from the compressive fibre.

3. Straight line stress distribution for a homogeneous material

A_o = net area of the concrete section increased by n times the areas of the unstretched reinforcements A_{su} and A'_s.
A_c = net area of the concrete section.
A_t = total area of the section, i.e. A_o plus n times the stretched reinforcement A_{sp}.
e_{1o} = distance of outside lower fibre from the centre of gravity of A_o.
e_{2o} = distance of outside upper fibre from the centre of gravity of A_o.
e_{1t} = distance of outside lower fibre from the centre of gravity of A_t.
e_{2t} = distance of outside upper fibre from the centre of gravity of A_t.
f_{1t} = concrete stress *) at the lower outside fibre at transmission of the prestress, psi.
f_{1w} = concrete stress *) at the lower outside fibre at working load, psi.
f_{2t} = concrete stress *) at the upper outside fibre at transmission of the prestress, psi.
f_{2w} = concrete stress *) at the upper outside fibre at working load, psi.
f_{wc} = concrete compressive stress due to working load alone, psi.
f_{wt} = concrete tensile stress due to working load alone, psi.
f_r = concrete tensile stress at first cracking, psi.
f_{rr} = concrete tensile stress, occurring at cracking after repeated loading, psi.
I_o = moment of inertia computed in accordance with A_o.
I_t = moment of inertia computed in accordance with A_t.
S_{1o} = sectional modulus related to fibre e_{1o}.
S_{1t} = sectional modulus related to fibre e_{1t}.
S_{2o} = sectional modulus related to fibre e_{2o}.
S_{2t} = sectional modulus related to fibre e_{2t}.

*) + denotes compression, − denotes tension.

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Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1945 for publication in the JOURNAL for June 1945.

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400 SECOND BOULEVARD, DETROIT, MICHIGAN (Supplement) November, 1945

Discussion of a paper by Paul William Abeles:

Fully and Partly Prestressed Reinforced Concrete*

By K. P. BILLNER, L. COFF, K. HAJNAL-KONYI and AUTHOR

ERRATA

p. 181, Synopsis line 5: change "formula" to "formulas."
p. 186, line 13: change "wire bonded" to "wire, bonded".
p. 192, last but one line: change "Δ_{s1}" to "Δp_{s1}".
p. 193, l. 3, formula (1a): change "p_s" to "Δp_s".

p. " 2 l. above formula (3b): change "$\frac{D}{d} = 1.1$" to "$\frac{D}{d} = 1.16$".

p. " 2 l. above formula (4a): change "p_i" to "P'_i".
p. " bottom l., formula (4a): change "$P_i.e'_{so}$" to "$p'_i.e'_{so}$".

p. 194, top l., formula (4b): change "$P_i + P_s$" to "$P_i + P'_i$".
after formula (4b) insert: "Formulas (3) and (4) are safe approximations. The exact values are obtained by formulas (3') and (4'), if P_i, P'_i, p_i and p'_i in formulas (3) and (4) are replaced by P_t, P'_t, p_t and p'_t respectively, as e.g.

$$\Delta p_s = \frac{n.P_t}{A_o}\left(1 + \frac{e^2_{so}}{g^2}\right) \quad \ldots\ldots (3')$$ "

l. 3 below formula (4b): change considrced" to "considered".
l. 4 from bottom: change "$I_i = I_o + A_o.\ (e_{1o} - e_{1i})^2$" to "$I_i = I_o + A_o.\ (e_{1o} - e_{1i})^2 + n.A_s.e^2_{si}$".

p. 195, change formula (8) to "$A_s = \frac{M_w.s}{j.d.f_{su}}$" i.e" $A_s = \frac{M_{w.s}}{j.d.f_{su}}$".

l. 4 below formula (8): change "The" to "Mostly, the".

p. 196, formula (11) change "I_o" to "S_{1o}".

p. 201, l. 2 below title: change "$2;j.d. = 12$ in." to "$2;\ j.d = 12$ in.".
l. 7: change "e_{ss}" to "e_{so}".
last l. before Table: delete "(No. 8)".
in Table l. 4 from bottom center column: change "1.07" to "1.01".

p. 208, Fig. 14: The widths of cracks are correct in inches but 10 times too great in mm. e.g. change "2.5 mm. = 0.01 in." to "Permissible width 0.25 mm. = 0.01 in."
l. 8 below Fig. 13 and 14: change "cracked ($n = 15$)." to "cracked section ($n = 15$)."

*ACI JOURNAL, Jan. 1945; *Proceedings* V. 41, p. 181.

p. 213, Add the following Notations:

"e_{s0} = distance of A_{sp} from the center of gravity of A_c.
e'_{s0} = distance of A'_s from the center of gravity of A_c.
e_{s1} = distance of A_{sp} from the center of gravity of A_t.
e'_{s1} = distance of A'_s from the center of gravity of A_t."

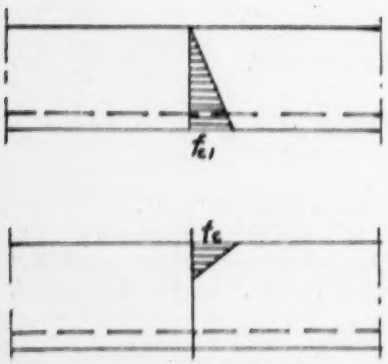
Reference 10.—: change "Forschingsarbeiten" to "Forschungsarbeiten."

By K. P. BILLNER†

Dr. Abeles' paper is praiseworthy for the thoroughness with which he manages to condense the history of prestressing into a few pages of type and yet bring out the essential features of many efforts. It is also noteworthy that while he describes his own contribution which constitutes another forward step in the right direction, he does not emphasize its merits.

There is, however, a statement regarding prestressing in general, which frankly, this writer does not understand, and which concerns the very core of the subject. This is the reference to tests by Freyssinet and by Hoyer (p. 189), supposed to prove that a reinforced concrete specimen will support the same ultimate load whether prestressed or not.

Consider the center of the span of a beam or slab of rectangular cross section. With prestressing, the stress distribution in the concrete is reversed so that the maximum compressive fibre stress occurs at its bottom before the live load is applied. (Fig. A.) When the design load is applied, the stress distribution is reversed so that the maximum compressive stress occurs at the top. (Fig. B.) With proper design, the concrete section in the prestressed beam will, therefore, carry a much heavier load. In general, the same cross section will be good for three times the live load which can be supported by the beam not prestressed. (Fig. C).



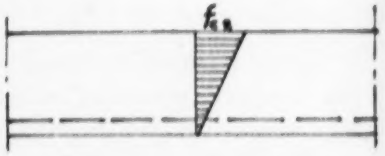


Fig. A, B, and C—Concrete stress diagrams

A (top left)—Form support removed.
B (top right)—f_{c2} stress from live load only.
C (left)—Conventional reinforcement—f_c—stress caused by dead load plus live load

†Vacuum Concrete, Inc., Philadelphia, Pa.

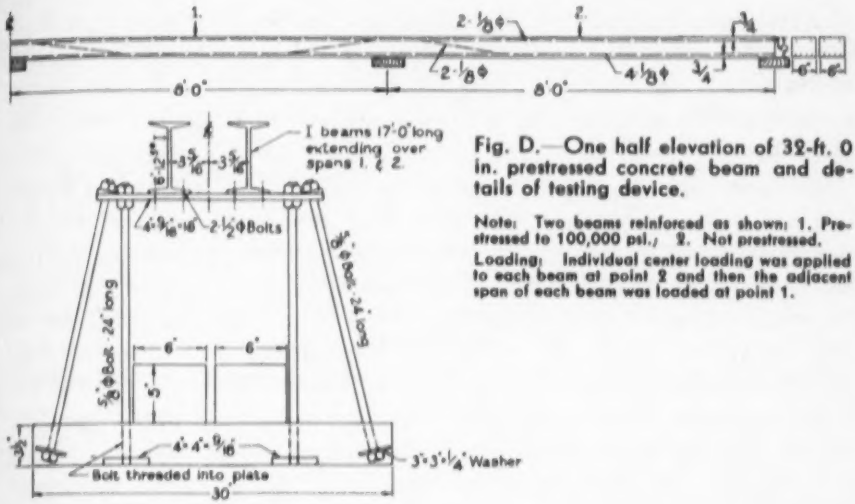


Fig. D.—One half elevation of 32-ft. 0 in. prestressed concrete beam and details of testing device.

Note: Two beams reinforced as shown: 1. Prestressed to 100,000 psi.; 2. Not prestressed.

Loading: Individual center loading was applied to each beam at point 2 and then the adjacent span of each beam was loaded at point 1.

The statement referred to holds good only if the percentage of prestressed reinforcement is very small. But then there is a waste of concrete and weight and the specimen is not a reinforced concrete beam but is rather a small suspension bridge.

Among the tests this writer has made one, intended to prove something else, may illustrate this point. Two identical concrete beams continuous over three intermediate supports were cast simultaneously. Each beam had a rectangular cross section area the height being 5 in. and the width 6 in. It was desired to ascertain the effect from prestressing as compared to a non-prestressed identical specimen. The arrangement for testing is shown in Fig. D. The reinforcement in each beam consisted of six ⅛ in. wires, two of which were bent up at the intermediate supports. The yield point of the wires was approximately 240,000 psi. The steel in one of the beams was electrically prestressed to 100,000 psi. after deducting loss from flow and shrinkage. Control cylinders cast of the same mix used for the beams, showed the concrete to have a compressive strength of about 5,000 psi.

One of the end spans of the prestressed beam was tested first. Failure occurred when the center load reached 2,700 lb. The beam then had a deflection of .37 in. in a span of 8 ft. No crack occurred at any of the supports. When the load was removed, the beam went back to nearly straight, with a permanent deflection after loading, of only two-hundredth of an inch at the center. The hair-crack which was visible under a maximum load, closed when the load was removed and could not be seen even by use of magnifying glass. The span adjacent to this end span was then loaded in a similar manner. It also took a center load

of 2,700 lb. at failure. The deflection on this span was then .3 in. and a hair-crack extending from the bottom of the beam one-half way up, was visible at the center. When the load was removed, this beam became almost straight, the permanent deflection being one-hundredth of an inch.

The next test was made on one of the end spans of the beams which had not been prestressed. The deflection at a center load of 700 lb. was 4/10 in. and there was a fairly wide crack extending from the bottom of the beam to nearly the top of it under the load. Another crack occurred at the intermediate support adjacent to it, caused by the negative bending moment there. When the load was removed, the beam recovered only part of its deflection; both of the cracks mentioned were still visible.

The adjacent intermediate span of the not-prestressed beam was then loaded. Failure occurred under a center load of 800 lb. The same kind of cracks as in the end span of the beam occurred. The beam recovered only part of its deflection when the load was removed and the cracks remained open.

An analysis of the tests indicate that under the maximum load, the steel in the prestressed beam was stressed to about 240,000 psi., whereas the stress at failure in the not-prestressed beam anounted to an average of 54,000 psi. It may be noted that in each case the loading was carried on until the deflections were about 50 percent greater than the so called permissible $\frac{1}{360}$ of the length of the span. The tests were witnessed by Prof. H. C. Berry.

By L. COFF*

Dr. Abeles points to the safe use of very much higher unit steel stresses, than customary in reinforced concrete construction so far. According to this paper, a steel stress of 100,000 psi. would not lead to dangerous cracks, if combined with adequate extreme fiber restraint through partial prestressing.

Drawn wire would be the material used for reinforcing, prestressed and non-prestressed.

While a stress of 100,000 psi. seems somewhat startling, in view of the small elongation concrete can withstand without cracking, our attitude of limiting steel stresses on this score should be revised in the light of actual experience with monolithic concrete construction.

Our wisdom is based, in the first instance, on theoretical considerations, combined with tests on single span beams, freely supported under

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ideal laboratory conditions. These conditions do not exist in actual practice. Even a beam bearing on brick walls will behave differently as to extreme fiber restraint. In monolithic reinforced concrete construction, there should be very little analogy with laboratory tests on single spans simply supported.

Better bond contributes to more effective fiber restraint, and should be examined in combination with other restraining factors.

It may be argued that frame action is taken care of by our computations, the maximum steel stresses being arrived at after considering the degree of restraint. How reliable are these computations? We figure deformations of the uncracked structure and stresses for the cracked section. When the section has cracked, our original assumptions are no longer true.

Besides, the elastic theory we are using has its limitations, one of them being moment redistribution. The tests of W. H. Glanville and F. G. Thomas,* generally recognized as an outstanding research performance, bear out that the elastic theory is correct up to the point, when in the critical section, either the concrete reaches its ultimate strength or the steel is stressed to the yield point. Thereafter, the structure does not collapse, as we would expect, but other sections help to maintain the moment of resistance of the critical section constant, up to a load often several times the one assumed in our computations to result in failure. In tests on elastic frames and continuous beams, ultimate loads of three to six times the computed ones, were reached.

We are living in a theoretical world, far away from what actually happens in a structure. Of this we are acutely aware when dealing with new methods or materials.

The writer was in this position in the early thirties, when, together with G. E. Clarkson, a Canadian metallurgical engineer, he played an active role in creating the industry of cold developing reinforcing steel in the British Empire. About 40,000 tons of reinforcing steel, at computed stresses of between 24,000 and 30,000 psi. were supplied and incorporated into concrete, while he was in charge of the technical part of this venture. Simultaneously, he had access to the experience gained with similar types of developed steel on the continent of Europe, to the extent of several hundred thousand tons.

More cracking should have been expected, than when using stresses of 16,000 to 18,000 psi. However, this was not the case. After two or three years of sales, the writer who believed in the theory, higher stresses, more cracking on the job, was advised by building owners, architects

*Section V of Studies in Reinforced Concrete, Technical Paper No. 22. Issued by H. M. Dept. for Scientific and Industrial Research, London, 1939.

and consulting engineers that there was less visible cracking in their buildings, than when using ordinary, plain reinforcing bars.

Similar advice came from Europe, especially from Switzerland and Czechoslovakia. In the United States the Wickwire Spencer Steel Co., had very favorable job results. In Europe, better bond may have had something to do with it. The writer is inclined to believe, however, that the main factors resulting in less visible cracking were, in the first instance, the absence of a definite yield point in cold developed steel, and the beneficial effect on shrinkage cracks, of low percentages of reinforcement. With regard to the yielding of steel, it seems there are in every reinforced concrete structure overstressed sections, in which the steel reaches the yield point. With cold developed steel, including wire, this need not necessarily lead to the opening of cracks as it does when the yield point is pronounced.

The most conclusive evidence that high steel stresses, computed by our usual methods, do not necessarily mean more visible cracking, has been gathered through an experience on such a gigantic scale, that it is worth recording.

Earl's Court Exhibition Buildings in London, built in 1935-37 by the Hegeman-Harris Co., of New York, under the personal direction of John W. Harris, is one of the largest reinforced concrete structures ever built, containing close to 100,000 cu. yd. of reinforced concrete, and more than 16,000 tons of reinforcing steel. It covers about 25 acres of construction area, in multiple story, wide span buildings, constructed to house trade exhibitions. The general contractors appointed their own architect and consulting engineers, amongst them the writer, who was responsible for the suspended car park and chair store.

Several years after completion of the buildings, the writer was called in by the building authorities to advise on the opening of a trade show, the Ideal Home Exhibition, a yearly London institution, with a normal attendance of 100,000 visitors a day. Unknown to the authorities, some exhibitors had erected a considerable number of two-story brick homes, with elevated rockery gardens around them, in the chair store, covering about 56,000 sq. ft.

The concrete floors in bays of 20 ft. x 30 ft., so loaded, had been designed in the writer's office for a live load not to exceed 200 lb. per sq. ft. The story under the floor so excessively loaded, was a suspended car park which had to be kept free of shoring. Several of the columns supporting this floor and the chair store were bearing on a steel girder bridge crossing a suburban railroad line.

The first investigation revealed that the steel bridge showed no increased deflection, notwithstanding the increase of load. Most likely

the rows of columns, with the very stiff floors on top and bottom, were supporting themselves like a Vierendeel System across the bridge, thereby relieving the column load. The two concrete floors nowhere showed greater deflections than 1/2000 of the span, nor could cracks of any consequence be discovered. A take-off and weight calculations of the brick homes and their surrounding gardens were made, and the structure checked as to stresses under the loads as described. It was found that the steel was stressed in places at more than 45,000 psi., which stress would have been increased to between 55,000 and 60,000 psi. by the crowds expected. The reinforcing steel was mainly Isteg steel tested at 54,000 psi. for a .2 per cent permanent set.

So many cases of overload capacity are known in actual practice, that the writer was not particularly worried about the safety of the structure, excepting for the steel bridge, which had to be dealt with separately. Eventually it was decided to allow the exhibition to open according to schedule, i.e., two weeks after this discussion took place, subject to testing the floors under an additional load of 100 lb. per sq. ft. in the most unfavorable positions. This test involved the handling of nearly 1000 tons of material and employing a considerable crew, measuring deflections, and looking for cracks, in revolving shifts. It was found that the deflection did not exceed 1/1500 and the cracks were negligible.

In the light of the above experience, it may well be that stresses of 100,000 psi., as mentioned by Dr. Abeles, should not be objectionable, especially if supplemented by additional extreme fiber restraint through partial prestressing.

Naturally, when comparing total and partial prestressing, the first sounds very attractive in view of the elimination of many uncertain assumptions in computing conventional reinforced concrete construction. There again, partial prestressing offers great advantages as to being able to comply easily with variation and alternation of moments.

For the time being, it remains to be demonstrated whether prestressing, either total or partial, will become a commercial proposition under American conditions, excepting for the use of cables in structures of considerable magnitude. For the handling of a multitude of individual wires, Schorer and Billner have shown a way. Other methods may follow.

The most frequent complaints against cracking in conventional reinforced concrete structures refer to shrinkage and diagonal tension cracks. In this respect, longitudinal ties with destroyed bond, according to Dill's method, would improve the shrinkage situation, while prestressed stirrups should contribute materially in reducing diagonal tension cracks. Prestressed stirrups, bonded around the tension bars, and with

coated legs, threaded at the ends for pulling up, are simple to apply. So are shrinkage bars with destroyed bond. Tests designed by the writer are in preparation at Toronto University to ascertain the effectiveness of such arrangements in continuous structures. The tests of Evans, mentioned by Dr. Abeles, with conventional bars, but prestressed stirrups, form a valuable precedent.

As already mentioned, prestressing, whether fully or partly, points to the use of drawn wire. In this connection the creep of wire under sustained load should not be overlooked. Freyssinet, as well as Hoyer, dealt with it. Within the proportional limit, wire, cold drawn to a specified tensile strength, is a perfectly stable material, as shown by creep tests over a period of nine years by the John A. Roebling Sons Co. The original proportional limit can be raised considerably through cold working. Hoyer mentions 85 percent of the ultimate strength as the proportional limit of the wire he uses. His tests, supporting this percentage, cover too short a test period to be conslusive. In the absence of standard specifications for prestressing wire, the characteristics of this material should be studied carefully in every case.

By K. HAJNAL-KONYI*

Although the tests on partly prestressed beams described in Dr. Abeles' paper were preliminary only and further tests on this type of construction are necessary, they revealed some important features of partly prestressed beams. The main objection to such beams, raised in a discussion referred to by Dr. Abeles under (9), was that under the design load "strong cracking" would exist. According to the test, in a beam with only 40 per cent of the reinforcement prestressed: 1) the cracks were very fine when they first occurred and the rate of increase was slow; 2) the "permissible" width was reached at a load of about 75 percent of the ultimate, i.e. far in excess of the permissible load; 3) even after increasing the load up to 80 percent of the ultimate, cracks of .02 in. width (i.e. twice the permissible limit) completely disappeared when the load was removed. If it can be proved that these findings may be generalized, partly prestressed structures should have a wide field of application.

The favorable results of these tests gave me the idea to combine precast prestressed and in-situ reinforced concrete, with or without additional reinforcement in the in-situ portion. I submitted this suggestion to the Institution of Civil Engineers in London in March 1944 for publication.†

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†Correspondence on "The Use of Pre-cast Pre-stressed Concrete Beams in Bridge Deck Construction" by Alan Andrew Paul. *Journal of The Institution of Civil Engineers*, London, Oct. 1944: Supplement.

The reasons for combining precast prestressed and in situ concrete may be summarized as follows:

Precast prestressed units are more expensive per unit volume of concrete than cast-in-situ concrete. If fully prestressed, they require careful handling unless special reinforcement is provided for this purpose and the very high compressive strength of the concrete cannot be fully utilized in the structure, since the maximum stress under working loads is smaller than the stress at the release of the stretching force at a comparatively early age of the concrete. When such precast units are assembled for a floor or bridge, they act as single units, do not form a monolithic structure without an additional screed on top of sufficient strength or transversal prestressing on the site, and cannot be used as components of a continuous beam or frame. All these disadvantages can be avoided if precast prestressed units are combined with in-situ concrete in such a way that they are united in a monolithic structure. This can be achieved by securing complete bond between the precast and in-situ components, e.g. by steel left protruding from the precast units or by shaping the precast units in a suitable manner, etc.

A few examples of possible applications may explain some of the main features of such combined structures. Fig. E shows the proposed cross section of a road bridge. In flat country it is often essential to keep the overall depth of a bridge as small as possible in order to reduce the length of the approach ramps and the area covered by these ramps. The use of prestressed concrete in itself allows a considerable reduction as against steel or cast-in-situ reinforced concrete. The arrangement shown in Fig. E makes a further reduction of the depth possible. The precast prestressed units must be strong enough to carry their own weight plus the weight of the in-situ concrete forming part of the structure plus any incidental loads during erection, but any additional load such as the road surfacing and the live load is carried by the combined structure. Thus the precast prestressed units can be lighter and need not be prestressed to the same extent as if they had to carry the full load. They serve as permanent shuttering and no scaffolding or temporary shuttering for the in-situ part of the structure is required. The in-situ concrete on top of the precast units increases the effective depth.

Fig. F and G show two alternative stress distributions under full design loads. The notations in these figures are as follows:—

f_{wd} = compressive concrete stress at the top of the precast units due to the dead weight of the units and the additional dead weight of the in-situ concrete.

f_{ww} = compressive concrete stress at the top of the precast units due to any additional dead load and live load.

f_w = maximum compressive concrete stress in the in situ part of the combined structure, due to any additional dead and live load.

f_{wd} is calculated on the section of the precast beams,

FIG. E

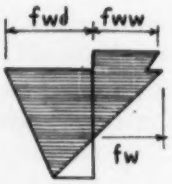


FIG. F

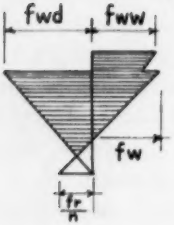


FIG. G.

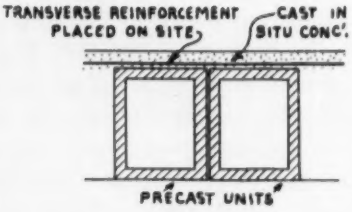


FIG. H

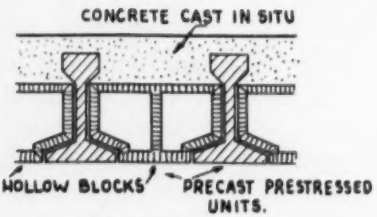


FIG. I

f_{ww} and f_w are calculated on the combined section.
f_t = tensile stress in the reinforcement, corresponding to f_{ww}
n = modular ratio for precast concrete.

The diagrams are based on the assumption of a smaller modular ratio for the precast concrete than for the in situ concrete.

Fig. F corresponds to a fully, Fig. G to a partly prestressed structure. The latter can be so designed that cracklessness under dead load is guaranteed. Apart from exceptional cases, this seems to be the most advantageous application of the suggested type of combined structure. Any fine cracks that might occur under heavy traffic will disappear immediately when the load is removed so that the bridge will remain free of cracks during the best part of its life. This is a great improvement on ordinary reinforced concrete bridges, the cracklessness of which under dead load can by no means be relied upon.

It should be noted that in such a structure, even if fully prestressed, the extreme fibre stress in the precast part under design load may substantially exceed the maximum concrete stress at the release of the

stretching force. At the same time, the maximum permissible stress of the in-situ concrete can be utilized, which, of course, is smaller than the permissible stress in the precast part.

In a structure with a cross section according to Fig. E and a stress distribution according to Fig. G, the reinforcement placed in the in-situ concrete remains unstressed under the weight of the structure. However, the whole of the main reinforcement may be accommodated in the precast part, if desired.

Fig. E shows a cross section without cavities. It is obvious that cavities can be arranged. The total dead weight of a combined structure need not be greater than that of a fully prestressed precast structure, but the overall depth which is governed by the maximum fibre stress of the concrete—apart from deflection—can be smaller. Fig. H shows an application with hollow precast units without additional main reinforcement on the site. A part of the reinforcement of these beams may not be prestressed. Fig. I shows that the precast units may be spaced and the gaps filled with hollow blocks. A further development on this line is the construction of a floor of the shape of an ordinary reinforced concrete floor in which the webs of the T- and L-beams are formed by precast prestressed units and the flanges by a slab cast in-situ.

The use of precast prestressed beams as webs of T-beams with in-situ flange is mentioned in an article by Soutter* in which the results of a test on such a beam are described. This test has proved that full co-operation between the precast and in-situ components can be achieved and it is possible to utilize fully the prestressed reinforcement which was broken at failure of the test beam.

The sectional area of the steel was 3.1 sq. cm., its ultimate strength 19.5 t per sq. cm. and the distance from the centre of gravity of the reinforcement to the middle of the flange of 10 cm. thickness was 39 cm. The maximum bending moment resulting from these figures, as calculated by the author, is 3.1x19.5x39 = 23.6 tm, the actual test result was (with two point loads at the third points of the span) 28.1 tm. This is an excess of 19 per cent. Even if we assume that the resulting compressive force was nearer to the top of the beam than assumed in the above calculation, this could only account for about half of the excess.

In my opinion, this excess has nothing to do with prestressing but is due to the same effect as observed in tests with square twisted steel and discussed in my paper referred to by Dr. Abeles' reference 11.

*P. Soutter: Die Verbundwirkung zwischen vorgespanntem und nicht vorgespanntem Beton und ihre Anwendung auf den Plattenbalken mit vorgespanntem Steg. Schweizerische Bauzeitung, Bd. 124 No. 8, 26 Aug. 1944. S. 103-108, No. 9. Sept. 2, 1944, S. 126.

AUTHOR'S CLOSURE

Errors in the paper are corrected at the beginning of the discussion. One point, however, is of great importance and is further discussed before considering the contributions to the discussion.

As pointed out, equations (3) and (4) are safe approximations, the exact values being obtained, if P_i, P'_i, p_i and p'_i are replaced by P_t, P'_t, p_t and p'_t respectively. The value p_t is given by formula (2a), similarly $p'_t = p'_i - \Delta p'_s - \Delta p_{st}$. Since the exact value of Δp_{st} is not known, depending on the quality of the concrete and the interval between its hardening and the transfer of the prestress, the replacement of p_t and p'_t by p_i and p'_i in the formulas (3) and (4) means a simplification, the difference from the exact value being relatively small for a high prestress p_i and small percentage p, which is the case when high strength steel is used. On the other hand, when p_w is obtained from formula (6), or P_w,P'_w, P_t and P'_t from equations (10) and (11), which may be the usual case, it is just as simple to use the exact formulas, when obtaining p_t and p'_t from the following equations (2a'), representing a combination of equations (1) and (2a):

$$\left.\begin{aligned} p_t &= p_w + \Delta p_{s2} + \Delta p_p \\ p'_t &= p'_w + \Delta p_{s2} + \Delta p'_p \end{aligned}\right\} \quad (2a')$$

In this case, however, safe values for Δp_{s2} and Δp_p have to be assumed. The values $\Delta p'_p$ and $\Delta p'_s$ may differ from Δp_p and Δp_s, owing to the different compressive stress.

If, however, the percentage p is high (i.e. if mild steel is used), then it is imperative to use the exact formulas (3') and (4'), e.g. (3a') corresponding to (3a):

$$\Delta p_s = n \cdot \frac{d}{D} \cdot \left[1 + 3 \cdot \left(\frac{2d}{D} - 1\right)^2\right] \cdot p \cdot p_t = c \cdot p \cdot p_t \quad (3a')$$

In this formula, let in the usual manner p_t be replaced by $p_i - \Delta p_s$, the more complicated, but better approximation (3a") is obtained.

$$\frac{\Delta p_s}{p_i} = \frac{c.p}{1 + c.p} \quad (3a'') \text{ instead of } \frac{\Delta p_s}{p_i} = c.p \quad (3a)$$

The difference between the original approximation (3a) and equation (3a") related to the latter is:

$$c.p - \frac{c.p}{\frac{1 \times c.p}{1 + c.p}} = c.p.$$

For $n = 8$ and $\frac{D}{d} = 1.16$ according to equation (3b), c is 17.5; thus the

relative difference of the loss for $p = 0.1$ percent is 1.75 percent (0.0172 instead of 0.0175), gradually increasing to 17.5 percent for $p = 1$ percent (0.149 instead of 0.175). It should be borne in mind that the value c increases for smaller ratios $\frac{D}{d}$ (e.g. it is 21.8 for $\frac{D}{d} = 1.1$); consequently the approximate formulas should be used only for percentages up to 0.3 percent to 0.4 percent. Such small percentages, however, will be the normal case, when high strength wire is used. Equation (3a′) is better used instead of (3a) or (3a″), where the loss Δp_{s1}, according to formula (2a), represents a considerable part of p_i; hereby the correctness is improved, even if Δp_{s1} itself is not correct.

The difference between various prestresses and stretching forces at different stages and between the loss, owing to elastic deformation, according to the exact formula (3′) is seen from Table A, relating to 7 different systems of rectangular beams, (each 3 in. wide and 10 in. deep, the reinforcement having a cover of 0.5 in.) i.e. (3d), (3a), (3c), (3b), (6a) pre-stretched and post-stretched, and (6b) according to Table 1. Each system is designed for a permissible bending moment of 50,000 lb. in. (equivalent to a beam having an unstretched mild steel reinforcement of two square bars 7/16 in., $p = 1.26$ percent, designed for $f_s = 18{,}000$, $f_c = 1000$ psi. and $n = 15$). Systems (3d) and (3a) are designed for mild steel reinforcement, the percentage being according to the recommendation for system (3d)*: $\frac{100 f_c}{f_s} = 2.78$ percent i.e. 2.2 times that required for the traditional design. However, even with such high a percentage, the initial prestress of system (3d) becomes so high that mild steel cannot be used. The reinforcement of all other systems is of high tensile wire, calculated according to equation (8) for $s = 2.5$, $j = 0.83$ and $f_{su} = 250{,}000$ psi. : $A_s = 0.65$ in.^{2} (5 wires gage 8), $p = 0.23$ percent. The first 4 systems are fully prestressed, the 3 examples system (6) are designed for 0.4 P_w, P_w representing the stretching force required in systems (3c) and (3b). In all cases $n = 8$. For the systems with pre-stretched reinforcement it has been assumed that $\Delta p_s + \Delta p_p = 25{,}000$ psi., of which $\Delta p_{s1} = 10{,}000$ psi., whereas for the systems with post-stretched reinforcement $\Delta p_{s2} + \Delta p_p = 15{,}000$ psi. Generally, these are safe assumptions. It may be repeated that exact values for these losses cannot be obtained. If, however, higher values of the losses are assumed than those in fact occurring, an additional safety of the effectiveness of the prestress is obtained. From Table A it is seen that on the assumption of such rather high losses, in systems (3d) and (3a) very high concrete stresses occur at transfer of the stretching

*See U. S. Patent No. 2,035, 977.

TABLE A

System (Table 1)		3*d*	3*a*†	3*c*	3*b*†	6*a*		6*b*
						A	*B*+	
		Fully prestressed				partly prestressed		
d	in.	9.18		9.44				
p	%	2.83		0.23				
A_s	in.²	0.78		0.065				
A_{sp}		0.78		0.065		0.026		0.065
A_{su}		—		—		0.039		—
A_o	in.²	29.22		30 *				
e_{1o}, e_{2o}	in.	$e_{1o} = 5.11$, $e_{2o} = 4.89$		$e_{1o} = e_{2o} = 5$ *				
e_{so}		4.29		4.44 *				
I_o	in.⁴	236		250 *				
S_{1o}, S_{2o}	in.³	$S_{1o} = 46.2$, $S_{2o} = 48.3$		$S_{1o} = S_{2o} = 50$ *				
A_i	in.²	35.46		30.46				
e_{1i}, e_{2i}	in.	$e_{1i} = 4.36$, $e_{2i} = 5.64$		$e_{1i} = 4.93$, $e_{2i} = 5.07$				
I_i	in.⁴	330.7[2]		258.5				
S_{1i}, S_{2i}	in.³	$S_{1i} = 75.8$, $S_{2i} = 58.6$		$S_{1i} = 52.4$, $S_{2i} = 51.0$				
p_w(*equ.*6)	psi.	6,660		120,000				48,000
p_i		21,660		135,000				63,000
P_w	lb.	5,200		7,800		3,120		
P_i		16,900		8,780		3,510		4,100

†post-stretched. *approximately (small A_s neglected).

TABLE A—continued from opposite page

TABLE A

System (Table 1)		3*d*	3*a*†	3*c*	3*b*†	6*a*		6*b*
						A	*B*+	
		Fully prestressed				partly prestressed		
f_{1w}	psi.	0				−572		
f_{2w}		+567		+547		+807		
f_{1t}		+2,148		+1,073		+429		+500
f_{2t}		−926		−487		−195		−229
Δp_e(*equ.*3′)	psi.	15,160	—	7,880	—	3,150	—	3,680
p_i		46,820	21,660	152,880	135,000	148,150	135,000	76,680
P_i	lb.	36,520	16,900	9,930	8,780	3,850	3,510	4,980
	%	368	170	100	88.5	39	35.3	50
$\frac{P_i}{P_w}$	—	7.03	3.25	1.27	1.13	1.24	1.13	1.60

force, by far exceeding the permissible stresses. In view of these higher concrete stresses, the loss Δp_p ought to be increased for these systems, which in turn would effect a further increase of the initial prestress required and thus of the concrete stresses. From this comparison it is seen that the systems, having a mild steel reinforcement, are not suitable for prestressed beams (Too great difference between P_i and P_w and too high concrete stresses at the transfer).

The difference between fully and partly prestressed reinforced concrete is seen from a comparison of systems (3 b,c) and (6). In the latter cases the concrete stresses at the transfer are much less than those under maximum live load, whereas with all fully prestressed systems, mostly the greatest concrete stresses occur at the transfer of the prestress, when the concrete is relatively young. In pre-stretched systems a reduction of the bond stress at the transfer is of great importance. This is obtained in system (6b), the bond stress being reduced to $\frac{63{,}000}{135{,}000}$ =

0.47 of that occurring in systems (3c) and (6a), corresponding to the respective p_t values.

Mr. Billner's introductory acknowledgement is most gratifying. His contribution is valuable in giving an opportunity to clarify some misunderstanding. His Fig. D is an interesting example. Mr. Billner compares the cracking of a prestressed beam with that of a beam, having an unstretched high strength reinforcement. It can be assumed that the expression "failure", occurring where the first test is described, should read "cracking". This can be recognized from the description that the beam became nearly straight on removal of the load. In a prestressed construction, first cracking undoubtedly occurs at a much higher load than in a traditional structure. Total absence of cracks, however, is required only for particular constructions such as tanks for liquids, barges etc. In normal constructions the maximum width of cracks should be limited to 0.01 in., as pointed out in the "Introduction". Reference may be made to Fig. 14 in which first cracking occurred in the fully prestressed beam (c) at a load also about 3 times that in beam (a), of conventional type, whereas the load causing cracks of a permissible width of 0.01 in. is more than twice as high. It is an outstanding property of prestressed concrete, independent of the degree of prestress, as compared with traditional construction, that cracks nearly totally disappear on removal of the load. In view of this property, the width of 0.01 in.—which relates only to permanent open cracks [24]*—does not, in fact, count in prestressed concrete, provided the excessive load occurs only temporarily and does not remain for a considerable time.

On the other hand, Mr. Billner's objection relates to the ultimate load. His Fig. A to C refer to compressive stresses. Failure, however, in a construction with a small reinforcement primarily is due to yielding or failure of the steel, as has been pointed out in chapter 4. Shortly before failure, the counter-action, due to prestressing, is discontinued in the surrounding of the section where the cracks open, as the counter-pressure at the normal tensile zone cannot be transmitted. Consequently, approximately the same stress f_{su} occurs in the steel, whether the construction is prestressed or not, provided in the latter case there is a sufficient shear and bond resistance. There might be only a slight difference in f_{su} in favor of a higher prestress, owing to the fact that in this case the bond in the immediate neighborhood of the cracks is destroyed to a lesser degree.

All structures have to be designed on the basis of a guaranteed safety against failure, consequently formula (8) has been put up, giving the minimum reinforcement required to ensure a factor of safety s, inde-

*See additional references, end of discussion.

pendent of the degree of prestressing. It must be borne in mind that, in a prestressed construction, first cracking can be deliberately delayed by an increase of the stretching force until such a stage is reached that cracking and failure occur at the same time as with a brittle material. This extreme case proves, how important it is to ensure a definite factor of safety against failure, independent of that against cracking.

On the other hand, it is evident that the behavior of a prestressed construction, whatever the degree of prestress, totally differs from traditional reinforced concrete, and Mr. Billner's statement that the former is capable of carrying about 3 times the load of the latter before "cracking", is fully justified, provided the stretching force corresponds to this factor. Moreover, it is undoubtedly an outstanding advantage that a structure, even if only slightly prestressed, can be re-used after being loaded up to 80 - 90 percent of the ultimate load, whereas any conventional reinforced concrete structure would have permanent deformations of such an extent that its re-use would be impossible.

Mr. Coff refers in his discussion to the strength of monolithic reinforced concrete frames, greatly increased by the plasticity of reinforced concrete. However, this is quite a different problem which will have to be investigated separately, as has already been done for steel structures([25]). In the present paper, fully and partly prestressed concrete beams have been compared with conventional beams on the basis of tests on simply supported beams. The writer's suggestion that a partly prestressed concrete structure, reinforced with high strength wire, should be designed for a permissible stress of 100,000 psi. is based on these test results and has nothing to do with the re-distribution of stresses.

The reduction of visible cracking, at a stage at which the steel stresses are still below the yield point, cannot result from the absence of a definite yield point, as Mr. Coff believes. It is obviously the increased bond that accounts for a reduced cracking, since in this case the connection between the reinforcement and concrete is destroyed only in the immediate neighborhood of the cracks, as pointed out before.

Dr. Hajnal-Konyi's contribution is very helpful in demonstrating the possibilities offered by partly prestressed structures. His remark regarding the cost per volume of precast concrete cast in place may require some further explanation. The costs of concrete products, manufactured in a factory, together with the costs of transport from the factory to the site and of placing them into position (C_F) are compared with the costs of concrete put immediately in place (C_P), assuming no separate shuttering and support to be necessary. Let $C_F = a.C_P$, the value of "a" will vary greatly for different countries. Where the costs of either transport or labor—or of both are very high, a may exceed 3, otherwise it may be

considered to vary between 1.5 and 3, and thus an economy is effected if a part of the fabricated concrete is replaced by concrete cast in place, even if the total concrete quantity is slightly increased which, however, need not occur.

In the following, an example of a road bridge over a railway (span 34 ft.) is shown. The construction consists of I-shaped fully prestressed beams (A_c = 170 sq. in.) and of a deck slab for load distribution (see Fig. J), similar to the design reported in the paper [22]. Two alternative designs (Fig. K and L) have been made in accordance with Dr. Hajnal-Konyi's proposition. The precast beams of reversed T-shape (A_c = 81 sq. in.) are fully prestressed to carry their dead weight and that of the concrete cast in place, the weight of the surfacing and of the live load being carried by the combined construction. In design Fig. K, an additional mild steel reinforcement is placed in the concrete, thus such a structure as a whole represents system (5) Table 1, the precast units themselves representing system (6b). On the other hand, in design Fig. L, according to system (6a) Table 1, the total high strength reinforcement is placed in the precast units, half of it being stretched. The unstretched wires are preferably arranged in pairs twisted, to increase the bond.

For a fully prestressed beam, generally, the following relation (14) between the depth D, the span L (both in ft.), the uniformly distributed load w (lb. per sq. ft.) and the stress f_{wt} is obtained from equation (13), r representing a rectification factor with respect to the Sectional Modulus of a rectangle of the width B (in ft.):

$$f_{wt} = \frac{\frac{B.w.L^2}{8} \times 12}{\frac{r.B.D^2}{6} \times 12^3} = \frac{w.L^2}{192\ r.D^2} \text{ psi} \quad \ldots\ldots (13)$$

$$\frac{D}{L} = \frac{1}{8}\sqrt{\frac{w}{3\ r.f_{wt}}} \quad \ldots\ldots (14)$$

The rectification factor r will vary greatly, say, from 0.5 for a reversed T-shape to 0.9 for an I-shape, approximating a rectangle. The value f_{wt} should not be too high, say 1100 to 1200 psi., since with full prestressing f_{1w} should be positive. According to Fig. 3 and 4 the following relation exists: $f_{1w} + f_{wt} < f_{1t}$, hence $f_{wt} < f_{1t}$, but should not be higher than, say, 1300 psi.

For f_{wt} = 1100 psi., the formula (14a) is obtained, resulting in (14b) and (14c) for r = 0.85, corresponding to an I-shape Fig. J, and r = 0.60, corresponding to reversed T-shapes Fig. K and L respectively.

$$D = \frac{L}{20}\sqrt{\frac{w}{528\ r}} \quad \ldots\ldots (14a), \quad D = \frac{L}{20}\sqrt{\frac{w}{450}} \quad \ldots\ldots (14b),$$

$$D = \frac{L}{20}\sqrt{\frac{w}{315}} \quad \ldots\ldots (14c)$$

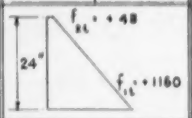
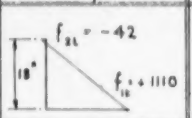
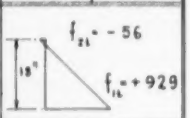
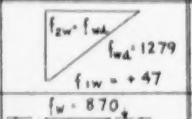
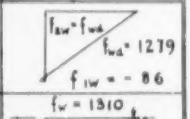
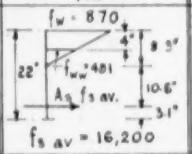
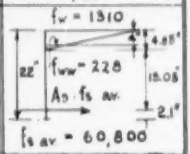
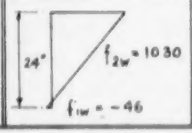
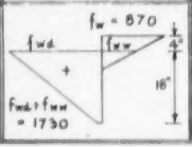
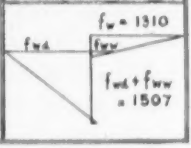
If the beams are provided at a distance B_o, the loading w has to be replaced by $\frac{B_o}{B}w$, B being the width of the rectangle for which r is obtained.

If w_o is the load per lin. ft. of the beam, w has to be replaced by $\frac{w_o}{B}$.

In the example, according to the British Regulations for Highway Bridges, the live load is replaced by a uniformly distributed load of 220 lb. per sq. ft. and by a knife edge load of 2,700 lb. per lin. ft. at the center. The dead weight of the construction Fig. J is about 250 lb. per sq. ft., thus the total load $w = 220 + \frac{2 \times 2.700}{34} + 250 = 630$ lb. per sq. ft., consequently $D = \frac{34 \times 12}{20}\sqrt{\frac{630}{450}} = 24$ in. On the other hand, for the beams of reversed T-shape, Fig. K and L, fully prestressed for the dead weight of 22 in. concrete, $w = 264$ lb. per sq. ft. and the depth required is: $D = \frac{34 \times 12}{20}\sqrt{\frac{264}{315}} = 18$ in.

In Table B, the individual stretching forces and prestresses are shown in addition to the stress distribution at the transmission and at loading. Different prestresses p_w and p'_w have been chosen for the 3 designs, consequently the stresses at transmission also differ. In design Fig. J, the same losses of the prestress have been taken into account as in the example Table A ($\Delta p_{s1} = 10{,}000$ and $\Delta p_{s2} + \Delta p_p = 15{,}000$ psi.). The latter value has been reduced to 10,000 psi. for the designs Fig. K and L, since at the stage when full prestressing has to be ensured (i.e. when the upper concrete is cast in place), only a part of Δp_p has taken place. However, it would not make a great difference, if the same amount i.e. 15,000 psi. were taken into account. For designs Fig. K and L, two loadings are distinguished: the dead weight (fully prestressed beams, $n = 8$) and the live load including the weight of the surface (combined construction, calculated in conventional manner, neglecting the concrete tensile zone, $n = 15$). When discussing the resulting stresses attained by addition of the two different stress distributions, according to the proposition of Dr. Hajnal-Konyi, it has to be borne in mind that full prestressing with respect to the dead weight remains effective even if some hair cracks might occur whenever the live load is applied. The resultant stresses at the top level of the precast beams exceed the conventional permissible concrete stresses. However, these stresses can be

TABLE B

FIGURE			J		K		L	
P_w	P'_w	p.s.i.	115,000		140,000	100,000	110,000	70,000
P_t	P'_t		130,000		150,000	110,000	120,000	80,000
ΔP_e	$\Delta' P_e$		8,360	1,100	8,200	300	6,900	100
P_i	P'_i		148,360	141,100	168,200	120,300	136,900	90,100
P_w	P'_w	lb.	72,200	18,000	39,700	6,300	34,600	4,400
P_t	P'_t		81,700	20,900	42,500	6,900	37,000	5,030
P_i	P'_i		93,000	22,100	47,600	7,580	43,000	5,660
STRESSES p.s.i.	AT TRANSMISSION							
	UNDER DEAD LOAD		—					
	UNDER LIVE LOAD (CONV DESIGN)		—					
	UNDER TOTAL WORKING LOAD							

considered as totally harmless, since they will increase much less than the stresses at the outside fiber of the combined structure, from where failure would commence.*

From the stress distribution in Table B it is seen that design Fig. K can be considered as a fully permissible solution, whereas with respect to design Fig. L an experiment might first be required to prove that in this case the same safety against cracking and failure is obtained as in design Fig. K, in spite of the high stresses in the stretched wire. In design Fig. K, the steel stress due to live load is in accordance with the permissible stress for mild steel, thus the additional stress in the stretched wire is less than the total losses of the initial prestress. In design Fig. L, however, the initial prestress of 136,900 is reduced to 110,000 psi.,

*If the different values of n for the precast and in-situ concrete are taken into account, the theoretical stresses at the top of the precast beams are even further increased, but this does not affect the actual safety, which depends on the in-situ concrete.

but the additional stress amounts to $\frac{15.9}{15.05} \times 60{,}800 = 64{,}200$ psi., thus totalling 174,200 psi., whereas in the unstretched wires the stress is only $\frac{14.2}{15.05} \times 60{,}800 = 57{,}400$ psi., further reduced by about $8 \times 1000 = 8000$ psi. pre-compression. In spite of the great discrepancy of these two stresses, about the same safety against failure is obtained as with the fully prestressed design Fig. J. This has been demonstrated by comparative similar tests discussed in this paper. A re-distribution of stresses takes place when the bond in the neighborhood of cracks is destroyed, thus discontinuing in that surrounding the effective prestress in the steel and the pre-compression of the concrete tensile zone. Since the shear stresses in the combined structures Fig. K and L are low, there is no reason why such an experiment should give different results from those discussed in this paper. If, however, a separate shear reinforcement is provided in the webs of the precast beams, the dead weight can be considerably reduced, as e.g. in the design Fig. M, an alternative to Fig. L. By increasing the depth of the concrete slab from 4 in. to 5 in., the stress at the top fibre is reduced by 100 psi. In this solution the weight to be carried by the precast beams is only 53 percent of that in Fig. L. The reinforcement A_s could be reduced by about 20 percent, but if the same A_s is provided as in Fig. L and the effective prestress is reduced to about 2/3 ($p_w = 70{,}000$ and $p'_w = 45{,}000$ psi.), a stress distribution is obtained, as indicated in Fig. M. Also the steel stresses are reduced in this case, as compared with Fig. L. The connection of the slab, cast in place, with the precast beams is effected by links, serving as shear reinforcement and protruding from the beams into the slab. Table C contains in detail the calculation of the design Fig. K, the cross section being shown in Fig. N.

In order to compare the designs Fig. J with Fig. K—M, the cost of the concrete per square foot is calculated on the assumption that in the equation $C_F = a.C_P$, a equals 3. Thus for the design Fig. J the concrete costs are: $(3 \times \frac{170}{144} + \frac{4}{12}).C_P = 3.87\ C_P$, whereas for the designs Fig. K and L the comparative costs are $(3 \times \frac{81}{144} + \frac{264\text{-}81}{144})C_P = 2.96\ C_P$. This is further reduced in the design Fig. M to $(3 \times \frac{81}{144} + \frac{5}{12}).\ C_P$*

*The cost of shuttering for the slab between the top flanges of the precast beams (e.g. by a precast thin slab) has not been taken into account, since it is only of slight influence. Moreover, it would be possible to use I-shaped precast beams in such a case, thus dispensing with any shuttering for the slab.

TABLE C

	$A_o = 18 \times 2 + (5 + 10) \times 2.5 + 2 \times 2 \times 2.5 \times \frac{1.5}{2} = 81$ in.^{2}
Net	$\Delta_o = \frac{1}{81} \times 5 \times 2.5 \times 7.75 = 1.19$ in.
area	$e_{1o} = 7.81$ in., $e_{2o} = 10.19$ in., $e_{so} = 6.56$ in., $e'_{so} = 8.94$ in.
A_o*)	$I_o = \frac{2 \times 18^3}{12} + 36 \times 1.19^2 + \frac{15 \times 2.5^3}{12} + 25 \times 6.56^2 + 12.5 \times 8.94^2 + 3.75 \times (4.81^2 + 7.19^2) = 3402$ in.^{4}
	$S_{1o} = 435$ in.^{3}, $S_{2o} = 333$ in.^{3}
	$A_t = 81 + 7 \times (0.283 + 0.063) = 83.42$ in.^{2}
Total	$\Delta_t = \frac{1}{83.42} \times [5 \times 2.5 \times 7.75 + 7 \times (0.283 - 0.063) \times 7.75] = 1.30$ in.
	$e_{1t} = 7.70$ in., $e_{2t} = 10.30$ in.
area	$I_t = \frac{2 \times 18^3}{12} + 36 \times 1.3^2 + \frac{15 \times 2.5^3}{12} + 25 \times 6.45^2 + 12.5 \times 9.05^2 + 3.75 \times (4.7^2 + 7.3^2) + 7 \times 0.283 \times 6.45^2 + 7 \times 0.063 \times 9.05^2 = 3559$ in.^{4}
A_t	
	$S_{1t} = 462$ in.^{3}, $S_{2t} = 345$ in.^{3}
	$P_w = 0.283 \times 140{,}000 = 39{,}700$ lb., $P'_w = 0.063 \times 100{,}000 = 6{,}300$ lb.
Stresses	$M_d = \frac{264 \times 34^2}{8} \times 12 = 458{,}000$ lb. in.
under dead	$f_{1w} = -\frac{458{,}000}{462} + \frac{39.700 + 6{,}300}{81} + \frac{39.700 \times 6.56 - 6.300 \times 8.94}{435} = + 47$ psi.
weight	$f_{2w} = f_{wd} = + \frac{458{,}000}{345} + \frac{46{,}000}{81} - \frac{260{,}400 - 56{,}300}{333} = + 1279$ psi.
Stresses at transmission of the prestress	$P_t = 0.283 \times 150{,}000 = 42{,}500$ lb., $P'_t = 0.063 \times 110{,}000 = 6{,}900$ lb.
	$f_{1t} = + \frac{42{,}500 + 6{,}900}{81} + \frac{42{,}500 \times 6.56 - 6{,}900 \times 8.94}{435} = 610 + 500 = +1110$ psi.
	$f_{2t} = + \frac{49{,}400}{81} - \frac{278{,}800 - 61{,}700}{333} = +610 - 652 = -42$ psi.
Initial Prestress	$\Delta p_e = 8 \times (610 + 500 \times \frac{6.56}{7.81}) = 8{,}240$ psi. $\Delta' p_e = 8 \times (610 - 652 \times \frac{8.94}{10.19}) = 304$ psi. } according to *equ.*(4′)
	$P_i = 0.283 \times 168{,}240 = 47{,}600$ lb., $P'_i = 0.063 \times 120{,}300 = 7{,}580$ lb.

*The exact value of e_o and S_o differ only in the decimals, if the relatively small values $A_s = 0.283$ in^2: and $A'_s = 0.063$ sq. in. are deducted.

TABLE C—continued from opposite page

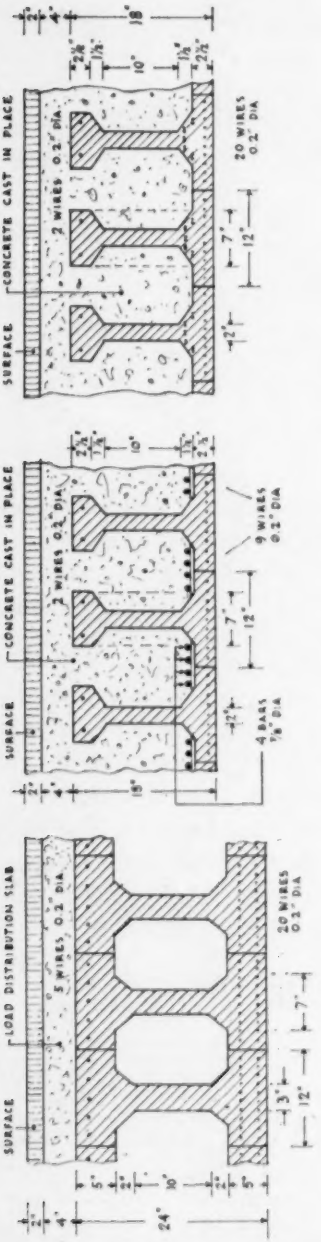
	$M_l = \underset{699{,}000 \text{ lb. in.}}{\frac{220 \times 34^2}{8} \times 12 + 2700 \times \frac{34}{4} \times 12 + \frac{24 \times 34^2}{8} \times 12} =$
Stresses under live load and under the weight of the surface	$d = 22 - \frac{1}{2.68} \times (2.4 \times 3.31 + 0.28 \times 1.25) = 18.9$ in.
	$p = \frac{2.4 + 0.28}{12 \times 18.9} = 0.0118$
	$k.d = 0.44 \times 18.9 = 8.3$ in.
	$f_s = \frac{699{,}000}{2.68 \times (18.9 - \frac{8.3}{3})} = 16{,}200$ psi.
	$f_w = \frac{2 \times 2.68 \times 16{,}200}{12 \times 8.3} = 870$ psi., $f_{ww} = \frac{4.3}{8.3} \times 870 = 451$ psi.

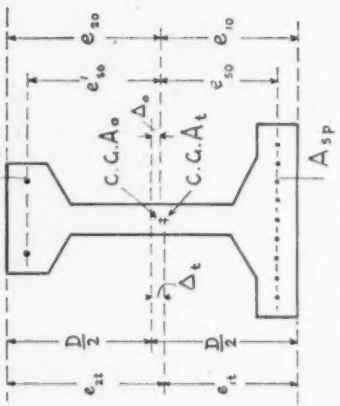
The different costs of the reinforcement have not been taken into account. In the Fig. J to M, only the longitudinal reinforcement is shown. About the same transverse reinforcement is required in all the designs to ensure either a load distribution (Fig. J) or a connection between precast beams and concrete in place (Fig. K—M). The amount of the longitudinal reinforcement is increased only in design Fig. K. The cost of prestressing is reduced in the alternative designs Fig. K—M, since instead of 25 wires only 11 or 12 wires respectively have to be stretched. The amount of links is reduced in the beams Fig. K and L as compared with Fig. J, but increased in the design Fig. M, in which, however, a reduction of the longitudinal reinforcement is possible.

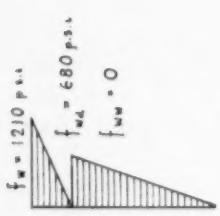
Table D shows a comparison of constructional depth, of cost of concrete per square foot expressed in relation to the price C_P per cu. ft., and of the total initial stretching forces required. It is seen that all the values relating to Fig. J are reduced in the designs Fig. K—M.

TABLE D—COMPARISON OF DESIGN FIG. J WITH FIG. K-M

Figure		J	K	L	M
Constructional Depth	in.	28	22		23
	%	100	79		82
Cost of Concrete	C_P	3.87	2.96		2.10
	%	100	76		54
$P_i + P'_i$	lb.	115.100	55.180	48.660	33.900
	%	100	48	42	29







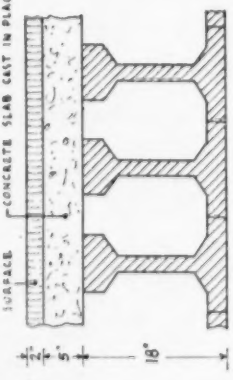


Fig. J, K, L (left to right at top)
Fig. M (precast beams as in Fig. L) and N (left to right at bottom).

Finally it may be added that, in certain circumstances, in a post-stretched construction p_t may differ from p_i, if the reinforcement is prevented from extending freely, owing to friction, resulting in a loss Δp_f (see equation 2b') [26]:

$$p_t = p_i - \Delta p_f \quad \ldots\ldots\ldots\

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An Instrument and a Technic for Field Determination of the Modulus of Elasticity, and Flexural Strength, of Concrete (Pavements)*

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Members American Concrete Institute

SYNOPSIS

An instrument for determination of the dynamic modulus of elasticity of concrete, in situ, is described. Test results are presented which show (a) the comparison of test values of E, obtained by various older methods, with that obtained with the new instrument; and (b) the relationship of such values to the flexural strength of concrete. It is concluded that adoption of the new method and technic is justified; and that widespread use of the new instrument would eliminate the necessity for casting field specimens during construction (except perhaps for day-to-day control purposes) or of removing costly "samples" from completed works. A method for determining the thickness of concrete pavements is briefly discussed. A rather extensive bibliography is included.

INTRODUCTION

The past few years have witnessed considerable advancement in nondestructive testing technics; with the so-called "sonic" (or perhaps more correctly "dynamic") methods of obtaining the modulus of elasticity of concrete, coming into increasingly widespread use. Powers [1] Hornibrook [2], and Obert [3] have employed values so obtained to reveal (a) the comparative strength and quality; (b) the resistance to frost action; and (c) the internal stress condition, of concrete. In virtually all of the work done by these investigators, however, laboratory specimens of comparatively small dimensions have been employed and E has been measured by flexural vibration methods. The authors of the present paper have, likewise, been much interested for a number of

*Submitted to the Institute August 24, 1944. This development was reported briefly by Mr. Long at the Research Session of Committee 115, at the ACI Convention Chicago, Mar. 2, 1944.
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[1] [2] [3] See Bibliography at end of paper.

years in the possibilities of non-destructive testing, and have applied dynamic technics to study of the elastic and plastic behavior of concrete. They have believed, however, that the full potentialities of such methods could never be realized so long as only laboratory specimens could be tested. Some instrument and method must be devised, which would be independent of size and shape of specimen, and would be applicable to actual concrete pavements (or other structures and materials) in the field. Specimens cast during construction, and more often than not, fabricated, cured and treated in a manner totally dissimilar to that of the actual field structure, were considered inadequate to reveal the true characteristics of the concrete pavement or structure supposedly represented.

The instrument and technic described in this paper, and the work which lead to their present development; constitute the authors' answer to the challenge presented by this situation; and it is hoped that the test results included will be helpful in clarifying the widespread speculation and misunderstanding concerning the practical value of "Dynamic *E*" determinations, in the field of engineering design and structural sufficiency evaluations.

METHODS OF OBTAINING DYNAMIC "E"

The most commonly employed method of obtaining dynamic *E* at the moment, is probably the flexural (resonance) vibration method used by Powers, Hornibrook, Obert and Duval, the authors, and many others. It has been amply described in the literature and will not be further expanded herein. Its limitation lies in that it is applicable only to specimens (usually prisms) of comparatively small dimensions.

Another method consists in determining the longitudinal frequency of vibration of similar specimens. In this case the limitation imposed by the method is the same.

Still another method (mentioned by Powers in a paper published in 1937, and since used by the authors, Obert, and perhaps others) consists in obtaining *E* by means of measuring the velocity of vibration (wave) transmission through the concrete. Since the velocity is nearly independent of size and shape of specimen, this latter method is not subject to the limitations imposed by the other methods mentioned, and hence has been the principal subject of experimentation of the authors and constitutes the principal matter of the present paper.

EXPERIMENTAL METHODS EMPLOYED

Three possible methods of measuring velocity have been investigated; involving two distinct approaches to the problem. These methods are

(a) the use of a source of sustained vibration of constant frequency to initiate waves in the concrete; and of analyzing such waves by means of a vibration pickup and cathode ray oscillograph. (b) the use of a single impact (hammer blow) to initiate a longitudinal, or compressional wave in the concrete, and measuring its velocity by means of a seismograph-type, time-recording instrument. (c) the use of a single impulse (as in (b)), but measuring its velocity by means of an electronic device consisting of two pickups, two thyratron tubes, and a triode-ballisitc galvanometer circuit. The first and last of these methods are illustrated by Fig. 1 to 4.

Selection of Present Method—(Electronic Interval Timer)

The method of (b) was abandoned because of its inadaptability to determination of wave velocity over short distances, and because of the cumbersomeness of the required film, and lack of precision in adequately recording short time intervals. The method of (a) has been superseded (at least for the present) by the simpler and more direct method of (c), yet it appears to be of sufficient importance, from the point of view of determining pavement thickness, as well as purely theoretical considerations of the wave phenomena involved, to warrant inclusion herein. A brief description of its employment to determine pavement thickness, in conjunction with the method of (c) is contained in a later section hereof.

Wave Types Involved in Test Methods

Theoretically, in the interior of an isotropic elastic solid, only two types of waves are possible [11] (a) a longitudinal (compressional) wave, in which the particles vibrate in a direction parallel with the direction of wave progression; and (b) a transverse (shear) wave, in which the particles vibrate in a direction perpendicular to the direction of wave progression. The velocities of these two types of waves are expressed as follows, using in general the notation of Timoshenko:

$$V_1 \text{ (longitudinal)} = \sqrt{\frac{\lambda + 2\mu}{\rho}} = \sqrt{\frac{E\,(1-\nu)}{\rho(1+\nu)\,(1-2\nu)}} \quad \ldots\ldots\ldots\ldots (1)$$

$$V_2 \text{ (transverse)} = \sqrt{\frac{\mu}{\rho}} = \sqrt{\frac{E}{2\rho\,(1+\nu)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where

λ = Lame's constant $= \dfrac{\nu E}{(1+\nu)\,(1-2\nu)}$

μ = coefficient of rigidity $= \dfrac{E}{2(1+\nu)}$

E = modulus of elasticity

ν = Poisson's ratio

ρ = density

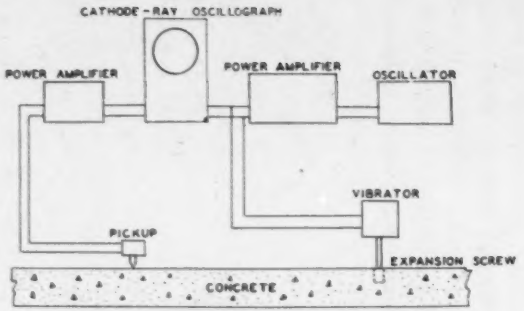

Fig. 1—Schematic diagram of velocity test apparatus—sustained vibration method

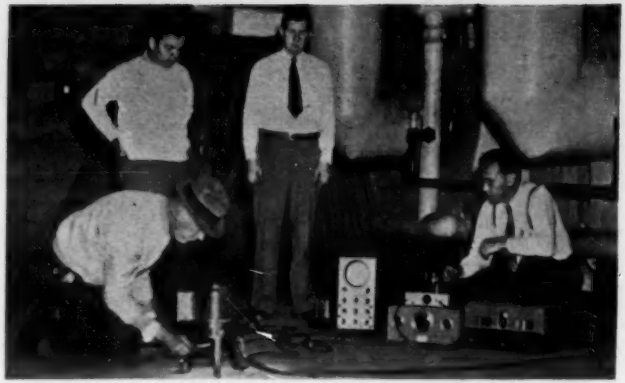

Fig. 2—Sustained vibration apparatus

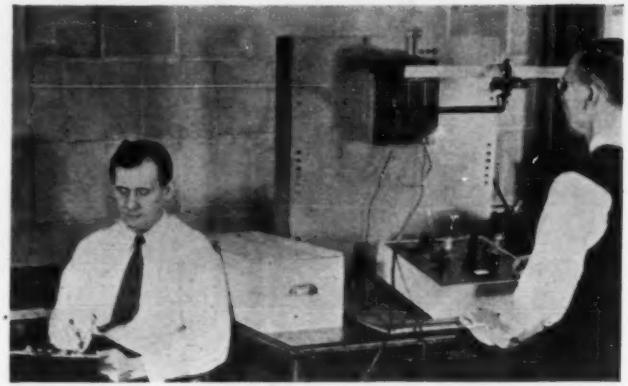

Fig. 3—Electronic interval-timer apparatus

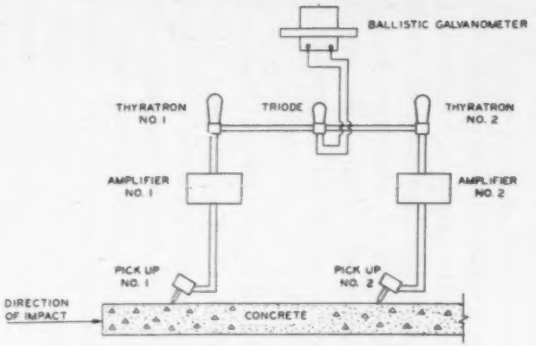

Fig. 4—Schematic diagram of velocity test apparatus—single impact method

The mathematical derivation of those relationships has been presented by Timoshenko [8], Macelwane & Sohon [11], Love [9] and others.

In elastic solids which are *not* infinite in extent, a number of wave types (and combinations thereof) are possible; however, for the sake of brevity and for the immediate purpose of this paper, only one additional type, the "transverse *vibration* wave" need be considered. This wave is present in elastic prisms (such as concrete pavements) in which the thickness is small compared to width and length, and should not be confused with the ordinary transverse (shear) wave previously referred to. If, in the analysis of the waves in such an elastic solid, the displacements are considered plane, then

$$(\lambda + 2\nu)\ (\nabla^2 \Delta) + \rho p^2\ \Delta = 0 \quad \ldots\ldots (3)$$

$$\nu \nabla^2 \omega + \rho p^2 \omega = 0, \text{ where} \quad \ldots\ldots (4)$$

$$\Delta = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y}$$

$$W = \tfrac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right)$$

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$$

If the boundaries are free to move, Timoshenko [10] has shown that the solution of these equations is

$$4\sqrt{(1-f^2)\ (1-h^2)}\ \text{Tanh}\ \frac{2\pi C}{l}\sqrt{1-h^2} = (2-h^2)^2$$

$$\text{Tanh}\ \frac{2\pi C}{l}\sqrt{1-f^2} \quad \ldots\ldots (5)$$

where

l = wave length of transverse vibration wave
$2C$ = thickness of prism
f = V/V_1
h = V/V_2
V = p/α, velocity of transverse vibration wave
V_1 = velocity of longitudinal (compressional) wave
V_2 = velocity of transverse (shear) wave
α = $2\pi/l$
p = $2\pi n$
n = frequency

Actually materials such as concrete are not perfectly elastic as assumed in deriving the equations; it is probable that both the modulus of elasticity and Poisson's ratio vary with wave-length and intensity of the wave source. For example, these so-called constants may vary with the

velocity and weight of traffic over a concrete pavement. In analysis of the test results presented herein, these facts should be borne in mind.

DESCRIPTION OF "ELECTRONIC INTERVAL TIMER"

The electronic interval timer (Fig. 4 and 5) consists essentially of (a) two similar, vibration-pickups and amplifiers; (b) two similar thyratron tube circuits, and (c) a triode-ballistic galvanometer circuit. (Fig. 6 shows a detailed wiring-diagram). The ballistic galvanometer used with this instrument is primarily designed for laboratory use, and is not too well adapted to field testing. A number of suggestions have been made to replace the galvanometer, the most promising of which appears to be a condenser and vacuum tube voltmeter, the operation of which has been described by Weisz [15].

Fig. 5—Electronic interval-timer field test

TEST PROCEDURE

The testing procedure, using the electronic interval timer to measure longitudinal wave velocity, is as follows: An impact (hammer blow) is applied to the concrete in a horizontal direction, approximately in line with the two pickups, thus initiating a wave impulse which actuates pickup number 1 and pickup number 2 in turn, as the wave travels through the concrete. The voltage generated in pickup number 1 is amplified and serves to ionize thyratron tube number 1 which, in turn, starts the flow of a constant current through the triode-ballistic-galvanometer circuit.

When the wave impulse reaches pickup number 2 the voltage generated is amplified and ionizes thyratron number 2 which, in turn, reduces

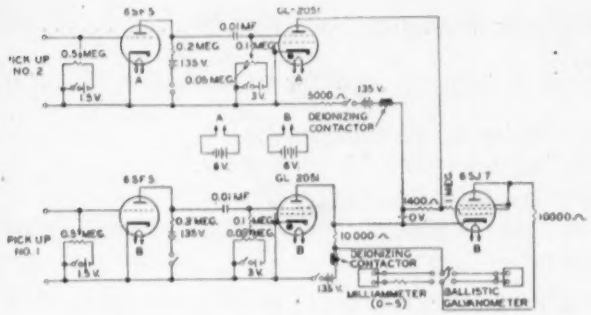

Fig. 6—Electronic interval-timer detailed wiring diagram

the current through the triode-ballistic-galvanometer circuit to zero. The effects of the wave impulse are then, (a) to start the flow of a constant current through the ballistic galvanometer when the impulse actuates pickup number 1, and (b) to stop the flow of this current when the impulse actuates pickup number 2. The deflection of the galvanometer is directly proportional to the time required for the wave to travel the distance between the pickups. The constant current is measured by ionizing thyratron number 1 and inserting a milliammeter in the triode-ballistic-galvanometer circuit.

COMPUTATION OF VELOCITY, AND MODULUS-OF-ELASTICITY

The velocity of wave transmission is computed from the formula,

$$V = \frac{I\,D}{g\,(m-m_o)}, \text{ in cm. per sec., where} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

I = current through galvanometer, in microamperes

D = distance between pickups, in cm.

g = galvanometer constant, in microcoulombs per millimeter

m = deflection of galvanometer, in millimeters, corresponding to D spacing of pickups.

m_o = deflection of galvanometer, in millimeters, which is the ordinate intercept of the straight line drawn through several points of D plotted versus m, where D-values are abscissa and m-values are ordinates.

The use of m_o in the formula eliminates the necessity of adjusting the time-constants of the two pickup circuits to precisely the same value in order to have the ordinate intercept equal zero, which would be necessary if m_o were omitted. Its use is advantageous in that it also eliminates errors in timing due to slow, unequal changes in the time constants of similar units in the two pickup circuits.

The modulus of elasticity of concrete is computed from the formulas, $E = V^2\rho$, in dynes per sq. cm. (for laboratory beam specimens), [8] . . . . (7)

$E = V^2\rho\,(1-\nu^2)$, in dynes per sq. cm. (for pavements), [13] ..... (8)

$$E = V^2\rho\,\frac{(1+\nu)\,(1-2\nu)}{1-\nu}$$ in dynes per sq. cm. (for mass concrete), [11] .. (9)

V = longitudinal wave velocity in cm. per second.
ρ = density of concrete, in grams per cubic centimeter.
ν = Poisson's ratio (assumed to be 0.16 for concrete).
The value of E can be expressed in pounds per square inch by multiplying dynes per square centimeter by 1.45×10^{-5}.

It will be noted that E has no correction due to Poisson's ratio, when determined from longitudinal wave velocity in laboratory beam specimens. This is explained by the assumption that in laboratory specimens, longitudinal strain is accompanied by lateral expansion or contraction thus retarding the wave, whereas, in mass concrete lateral displacements are suppressed [8] [11], causing the wave to travel at a slightly greater velocity. The situation represented by pavements is intermediate between those represented by beam specimens and by mass concrete [12], since the lateral displacements are suppressed in the direction of width but not in the direction of thickness. Assuming a Poisson's ratio of 0.16, the corrections reduce the E values by approximately 2.4 per cent and 6.1 per cent for pavement and mass concrete, respectively. It is obvious from these comparatively small correction values that they are not sources of appreciable error in the computed E values, and therefore do not, for the purposes of the present paper, require further exposition herein.

DISCUSSION OF TEST DATA

Table 1 presents test data of concrete representing several airfield runway pavements. Beams were sawed from the pavements and tested in the laboratory for dynamic E (flexural resonance method), static E, flexural strength (third point loading) and compressive strength (modified cubes). Adjacent to the location from which the beams were taken, tests of the concrete were made with the electronic interval timer for comparison with the dynamic E and flexural strength test values obtained in the laboratory. It should be emphasized here, that the concrete tested in the field was at a different temperature and moisture content than the corresponding concrete beams tested in the laboratory and for this reason the test values should not be in exact agreement. The laboratory beams were submerged in water at 70 F. for 7 days previous to testing, whereas the concrete in the field varied in temperature roughly from 50 to 100 F; and the moisture content was unknown.

The effect, of moisture at least, is of considerable significance since it has been determined, on the basis of laboratory tests conducted over

TABLE 1—COMPARATIVE TEST RESULTS OF CONCRETE PAVEMENTS*

Feature	(1) Density	Strengths		Lab. "E" Vaues		Velocity Values		
		(2) Flex.	(3) Comp.	(4) Dynamic	(5) Static	(6) Ft/Sec.	(7) "E"	(8) Flex.Str.
Army Airfield A								
Runway.....	2.49	850	7020	6.90	5.14	14,720	7.08	915
Apron.....	2.48	975	6890	6.97	5.42	14,930	7.28	960
Taxiway.....	2.46	800	5970	6.84	5.11	14,950	7.24	950
Army Airfield B								
Runway (a)....	2.53	820	9740	7.60	5.51	14,040	6.55	830
Runway (b).	2.50	970	6520	7.05	4.68	15,400	7.80	1050
Army Airfield C								
Apron.....	2.49	910	8140	6.89	5.09	14,680	7.10	920
Taxiway.....	2.42	830	6420	5.77	4.69	14,090	6.31	790
Army Airfield D								
Runway (a)....	2.46	745	6250	6.79	4.96	15,360	7.61	1010
Runway (b)....	2.47	700	5600	6.77	4.72	14,390	6.73	860
Army Airfield E								
Runway (a)....	2.40	800	5850	5.72	4.28	14,550	6.68	850
Runway (b)....	2.48	980	8820	6.88	5.12	14,450	6.89	890
Army Airfield F								
Runway (a)....	2.48	1015	8130	7.16	5.66	14,840	7.18	940
Runway (b)....	2.45	805	6630	6.33	4.66	14,260	6.56	830
Army Airfield G								
Runway (a)....	2.41	890	5920	6.60	4.72	13,700	5.96	730
Runway (b)....	2.43	970	7270	6.74	5.48	13,700	6.00	740
Runway (c)....	2.46	1010	7370	6.68	5.77	14,570	6.86	885
Army Airfield H								
Runway (a)....	2.47	885	7050	6.75	4.83	14,100	6.31	790
Runway (b)....	2.43	880	5660	6.26	4.72	13,890	6.27	785

*All values are averages of at least three determinations; the strengths are in psi. and "E" values are in millions psi.
(1) Determined from normal and submerged weights.
(2) Third point loading.
(3) Ends of beams, tested as "modified cubes".
(4) Flexural (resonance) vibration method.
(5) Tested in flexure; third-point loading.
(6) "Electronic Interval Timer".
(7) Computed from equations (6) and (8)
(8) Derived from curve Fig. 7.

a period of several years, that variations of E values, of the order of 12 to 15 percent may be expected between oven-dry concrete and saturated concrete. The effects of variations in temperature have not been studied to date, but it is probable that they are not so important as are those attributable to changes in moisture content; and, in all probability, moisture *gradient.*

Table 1 also presents flexural strength values corresponding to the E test values obtained with the electronic interval timer. These values were taken from the curve of Fig. 7.

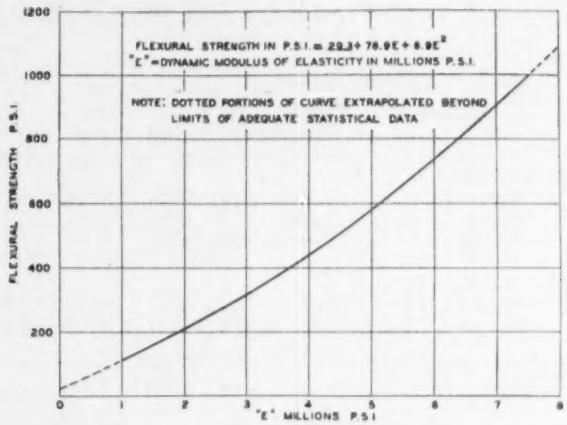

Fig. 7 — Relationship between dynamic modulus of Elasticity (E) and flexural strength of concrete beams

RELATIONSHIP OF *E* AND FLEXURAL STRENGTH

The curve of Fig. 7 was obtained from a statistical analysis of the dynamic *E* and flexural strength test values of approximately 1400 concrete beam specimens ranging in size from 4 by 4 by 16 in., to 8 by 9 by 40 in. However, about 1250 of these beams were of the 4 by 4 by 16 in. size. Dynamic *E* values were determined by the flexural resonance method; and flexural strength values were determined by the third point loading method. In the analysis of the data all of the test values were used, except for those falling in the extreme class intervals whose frequency was too low to be considered. In these instances the entire class intervals were disregarded. The effect of discarding the extreme (individual) values, which varied widely from the arithmetic mean, would have been to considerably decrease the "probable error", but would not have appreciably changed the formula for the curve of Fig. 7. The test data were divided into fourteen class-intervals of dynamic *E*, and the arithmetic mean was determined from the corresponding flexural strength values of each of these groups. The frequency of each class-interval used was twenty-five or more. The coordinates of the fourteen points thus determined were used in computing the constants of the second degree parabola [14] plotted in Fig. 7. This curve represents the relationship between dynamic *E* (flexural resonance method) and flexural strength, with a probable error of approximately 70 psi. It is possible then, to estimate with a reasonable degree of accuracy the flexural strength of concrete from laboratory dynamic *E* test values of beams, or from field test values of dynamic *E* obtained with the electronic interval timer on concrete, in situ.

It will be noted in analyzing the data that, in general, there is good agreement between the dynamic *E* test values obtained in the field and

those obtained in the laboratory. The greatest individual difference between the test values of the 18 field locations tested is approximately 17 percent. The average of all field values differs approximately 1.5 percent from the average of all laboratory values. A similar comparison, of the flexural strength values obtained from the field E values and the use of Fig. 7, and those obtained in the laboratory, shows that the greatest individual difference is approximately 36 per cent; and the average of all field determined values differs less than 1 percent from the average of all laboratory values. These values, shown in Table 1, demonstrate that there is a high degree of correlation between the flexural strength test values (column 2) and those obtained from the electronic interval timer E values (columns 7 and 8).

PAVEMENT THICKNESS DETERMINATION

In order to determine pavement thickness from the tests discussed above, it is necessary to know the wave length and velocity of the transverse vibration wave, and the velocity of the longitudinal wave. Fig. 6 presents a graph of Timoshenko's [10] "frequency equation" showing the relationship between, the ratio $\frac{(l)}{2c}$ of wave-length (of transverse vibration waves) to thickness, and the ratio ($t = V/V_1$) of transverse vibration velocity to longitudinal velocity, for two values of Poisson's ratio (0.16 and 0.25).

Since the ratio of V_1 to V_2 depends only on Poisson's ratio [11], h can be evaluated in terms of f. V and l can be determined from the sustained vibration test; and V_1 can be determined from the test with the electronic interval timer. This leaves but one unknown in the frequency equation; the thickness ($2c$), which can be computed.

For example; the following values l, V, and V_1 were obtained in actual tests of a concrete pavement nominally 6 in. thick: l = 2.55 ft. V = 3760 ft. per sec., V_1 = 12,520 ft. per sec. By making the required substitutions in the above formula, or by using Fig. 8, the value of thickness is found to be 6.3 in. Although it appears possible to determine the thickness of pavement to a reasonable degree of accuracy by the method described, the sustained vibration procedure of determining the wave length and velocity of the transverse vibration wave is rather tedious, and should be improved upon before it is generally adopted as a practicable routine test.

CONCLUSIONS

It is concluded (a) that the Electronic Interval Timer described herein is capable of measuring, with precision, velocities of wave transmission through concrete (or other elastic or semi-elastic materials), (b) that the

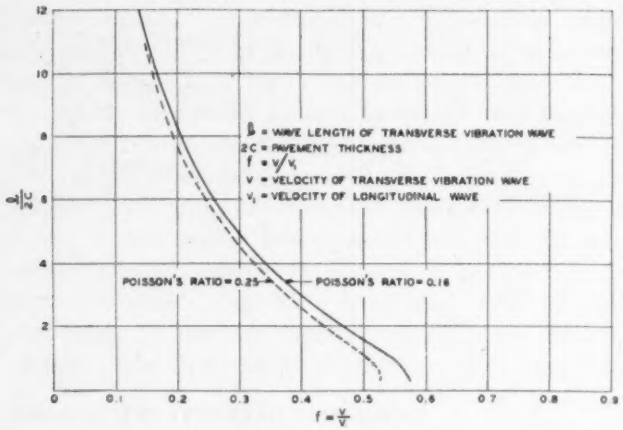
λ = WAVE LENGTH OF TRANSVERSE VIBRATION WAVE
2C = PAVEMENT THICKNESS
f = v/v₁
v = VELOCITY OF TRANSVERSE VIBRATION WAVE
v₁ = VELOCITY OF LONGITUDINAL WAVE
POISSON'S RATIO = 0.25
POISSON'S RATIO = 0.16
λ/2C
f = v/v₁

Fig. 8 — Graphical representation of Timoshenko's frequency equation

dynamic modulus of elasticity E, calculated from such velocity values, is quite as dependable as are moduli obtainable by any other method; (c) that the relationship existing between the flexural strength and dynamic modulus E of concrete has been shown to be sufficiently consistent to permit reasonably accurate predictions of flexural strength when dynamic E is known; and (d) that the method described forecasts the eventual abandonment of older, more laborious and time- consuming, and much more costly methods of obtaining knowledge concerning the flexural strength of concrete, in situ; and (e) that dynamic E values, although higher than the customary values obtained by static loading method, (being virtually dissociated from plastic flow or time yield effects) may prove to be more reliable and valuable than are the values usually used in design. This suggests the need for determinations of dynamic E and Poisson's ratio over a wide range of time, and loading (wave length and impulse intensity).

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Discussion of a paper by Long, Kurtz, and Sandenaw:

An Instrument and a Technic for Field Determination of the Modulus of Elasticity, and Flexural Strength of Concrete (Pavements)*

By PAUL L MORTON and ALEXANDER DODGE

By PAUL L. MORTON†

The authors are to be complimented upon their very ingenious method of measuring the elastic modulus of concrete *in situ;* unquestionably it is capable of yielding a great deal of hitherto unobtainable information. Through their kindness in supplying the details of their method even before its publication, the Engineering Department of the University of California was enabled to do experimental work along the same lines during the past year. Although we have not been able to realize the full possibilities of the method, we feel that further work on it is well justified.

To avoid the use of the ballistic galvanometer, which Messrs. Long, Kurtz, and Sandenaw found inconvenient, we built a timing unit in which the galvanometer was replaced by a capacitor which was charged during the interval to be measured. The charge on the capacitor was measured by means of a vacuum-tube voltmeter circuit, and the final time-interval readings were taken directly from a microammeter. By this means readings could be taken as rapidly as the data could be recorded. Fig. A of this discussion shows a front view and Fig. B a rear view of the interval timer, removed from its carrying case. It was operated from a 110-volt a-c power line, but could be built for battery operation if desired. (A similar timer is now available commercially, having 8 time ranges from 0.001 to 3 seconds, full scale.)

When the timer was used in the field it became apparent that the readings were more erratic than the timer, and the difficulty was traced to the type of pickup units used. The functions of the pickup units are to start

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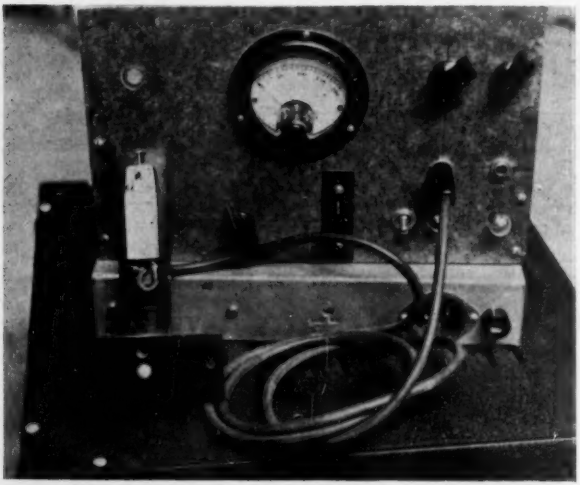

Fig. A—Front view time-interval meter, showing phonograph crystal pick-up.

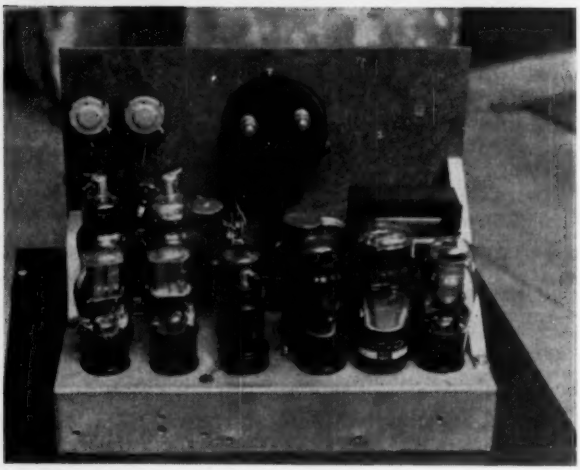

Fig. B.—Rear view time-interval meter.

the timing operation as the shock wave in the concrete passes the first or "start" pickup, and to stop the timing when the wave passes the "stop" pickup which might be from one to thirty feet distant. Following the authors, we first used vibration pickups consisting of a square flat slab of Rochelle salt supported at three corners, with the fourth corner free to flap or vibrate. This crystal was contained in a small casting which rested upon the concrete. The arrival of a shock wave moved the casting and started the vibration of the crystal. In a piezoelectric crystal of this type a voltage appears across the crystal whenever distortion occurs, and it was this voltage which was used to start and stop the timing operation.

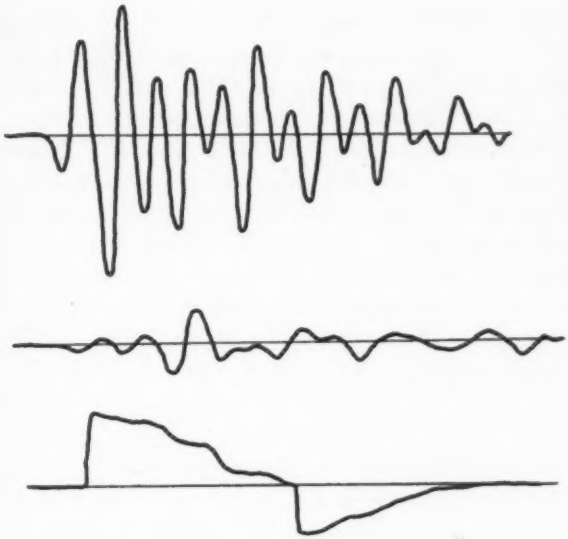

Fig. C, D, E (top to bottom) —C, Response of vibration pickup (shock wave from hammer blow—crystal 3 ft. from origin); D, Response to attenuated wave (same wave as Fig. C—identical crystal 12 ft. from origin); E, Response of phonograph crystal (shock wave from hammer blow as in C and D.)

In the type of crystal pickup just described it is the inertia of the crystal which is responsible for its distortion, since the free corner of the crystal tends to remain stationary when the crystal housing is moved by the displacement wave in the concrete. The voltage output of the pickup is therefore roughly proportional to the *acceleration* of the crystal housing, and does not reproduce the wave-form of the shock wave in the concrete. After the shock wave-front has passed, the crystal continues to flap at its own natural frequency, and the voltage output therefore resembles a damped sinusoid, modified by further motion of the concrete surface. Fig. C is a tracing of an oscillogram of the output voltage caused by a shock wave from a hammer blow on the concrete.

In Fig. C it will be noted that the voltage does not rise instantly when the shock wave arrives at the pickup, but only gradually. The instant when the timing unit starts will depend upon the sensitivity of the timing unit. Best results will be obtained if the timing unit is made extremely sensitive, so that it trips very early on the first half-cycle of the voltage. (Our unit, when set for maximum sensitivity, could be tripped by flipping the finger-nail upon the concrete near the pickup.) Note that if the timer fails to trip on the first cycle it may trip on a subsequent larger cycle. The delay between arrival of the shock wave and the tripping of the timer would not cause error if, but only if, both the start-pickup and the stop-pickup voltages operate the timer at the same point on the voltage wave.

Since the concrete is not a perfectly elastic medium a shock wave set up in it is gradually attenuated as it travels. Fig. D is a tracing of

an oscillogram of the voltage output of a second identical crystal pickup, subjected to the same shock wave which caused the voltage of Fig. C, but located 9 feet farther from the origin of the wave. The wave is greatly attenuated and somewhat changed in form; considerably more amplification would be necessary to cause the second pickup to stop the timing at the same point on the wave at which it started. Since the attenuation rate is unknown it would be difficult to preset the amplifiers correctly. Furthermore, it is impossible to set up wave trains of identical form repeatedly, because of the crumbling of the concrete where it is struck. In Figs. C and D the frequency of the oscillations is about 1500 cycles per second, and the shock wave travels about ten feet in the time of one cycle. Thus a slight difference in the points on the voltage cycle at which the timer starts and stops represents a considerable difference in the apparent velocity.

Obviously it would be an advantage to have the pickup unit supply a voltage proportional to the actual displacement of the concrete surface rather than to its acceleration, and to obtain a wave-front as sharp as possible. For this reason a phonograph pickup crystal was substituted for the vibration pickup; one of the phonograph units is plugged into the timer panel in Fig. A. The phonograph needle rested on the concrete surface, and this positive connection caused the distortion of the crystal to follow the motion of the surface and provide a proportional output voltage. The resulting output wave-form is shown in Fig. E, where the blow on the concrete was against a vertical surface. (The second step shows the arrival of a shock wave reflected from the far side of the slab.) For this type of voltage the timer will trip on the wave front or not at all, and even with a greatly attenuated wave at the second pickup the error is much reduced.

With the new pickups the kind of blow which initiates the shock wave becomes important, since the pickup will respond only to motion at right angles to the needle. Thus if longitudinal waves and transverse waves travel at different velocities the corresponding differences in transit time could be recorded. Experimental and theoretical study of the types of waves established by hammer blows or other kinds of excitation, and of the significance of the velocities of the different waves, should be very fruitful. Messrs. Long, Kurtz, and Sandenaw are to be congratulated upon opening up so promising a field for research.

By ALEXANDER DODGE*

The authors may be commended on the presentation of a matter which is always a subject of vital interest to the engineering profession, but the writer regrets that the exhibited material constitutes apparently

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only a minor portion of the extensive test data available. One notices, for example, that the flexural strength data in Table 1, if plotted on Fig. 7, will fall in the area corresponding to elastic modulus of 7,000,000 psi or greater, and one wonders if these data do not represent the highest of the fourteen class-intervals of dynamic E.

A conclusion may be drawn from the paper that the exhibited data represent approximately 5 per cent of the whole testing work performed. In such a case it would be of the utmost interest to know what the average flexural and compressive strengths are and the age of all 1400 concrete beam specimens, separately for 4x4x16 in. and 8x9x40 in. beams respectively.

The authors state that the curve in Fig. 7 represents the relationship between dynamic E and the flexural strengths, with "a probable error of approximately 70 psi." For the Army Airfield D, Runway (b), it is found, in Table 1, flexural strength of 700 psi for the dynamic E value = 6.77, and for the same value of E, the corresponding value of the flexural strength in Fig. 7 is 870 psi, or the difference of 170 psi, which is approximately 25 per cent greater than the actual strength.

The writer is of the opinion that it is premature to adopt dynamic E values for all design conditions. It has been discussed by many of the foremost engineers and proven by field experiments of others that the long time loadings, such as: seasonal and daily variation of the temperature, shrinkage and variation in moisture content in concrete as well as in underlying subgrade, may be, and very often is, the primary cause of failure of an unreinforced concrete; yet, just because of uncertainty and difficulty of analysis (and sometimes because of the resulting high stresses) these factors still are not given due consideration.

The writer's own studies indicate that, for the typical airfield construction, the stresses from these causes vary from 200 to 500 psi, and it is yet to be proven how much the high values of the flexural strengths (third point loading), shown in Table 1, might be changed by a long-time testing, say, of several hours, days and weeks duration.

These comments do not constitute a criticism of the fine work performed by the authors, but are offered, rather, as a warning to the profession at arriving at too hasty conclusions on the value of dynamic E values in design of concrete pavements.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Substantial temporary construction used for a naval training station

THOMAS H. CREIGHTON, *Engineering News-Record*, V. 133, No. 6 (Aug. 10, 1944), pp. 90-91.
Reviewed by S. J. CHAMBERLIN

The masonry block exterior walls are supported by a continuous reinforced concrete grade beam carried on capped wooden piles. The walls were reinforced above and below openings and at changes in section or framing to minimize cracking. The 8-in. concrete block were made in various special shapes and high pressure steam cured to speed construction. Scarcity of timber, availability of masonry labor and low maintenance were factors in the selection of masonry walls.

Resistance of concrete to freezing and thawing as affected by aggregates

STANTON WALKER, Director of Engineering, National Sand and Gravel Association.
Reviewed by the Author

Reprint of a paper presented before the Conference on Plans for Postwar Highways held at the University of Tennessee, Knoxville, May 12-13, 1944. Discusses general problem of concrete durability as related to aggregates and consists principally of summary of results of freezing and thawing tests of concrete by National Sand and Gravel Research Foundation at University of Maryland and made with a variety of fine and coarse aggregates from widely separated sources. Particular attention is paid to tests of concrete made with chert gravel under different conditions of moisture content. Effects of freezing and thawing on laboratory specimens were measured by determinations of dynamic modulus of elasticity.

Simplified concrete rigid frame design

JULIAN C. SPOTTS, *Engineering News-Record*, V. 133, No. 4 (July 27, 1944) pp. 80-83
Reviewed by S. J. CHAMBERLIN

Rigid frame concrete bridges with curved soffit decks are designed by first assuming relative proportions of the frame dimensions, completing the detailed analysis, and finally assigning true numerical values to the span, height and crown thickness. Four sets of charts are presented to simplify the design. The first set gives the coefficients for fixed-end moments, for uniform load, relative stiffness and carry-over factors for parabolically shaped fixed beams. The second chart gives the moment coefficients for parabolic loads due to the weight of the beam. The third set gives the negative moments for a moving concentrated load. The fourth set gives the elements of the fixed

beams (acting as the legs the rigid frame) for uniform side pressures, triangular earth pressure and for unbalanced sidesway. The procedure is illustrated by the design of a typical highway bridge.

Specifying controlled concrete

R. F. Moss, *Engineering News-Record*, V. 133, No. 6 (Aug. 10, 1944) pp. 86-89.
Reviewed by S. J. Chamberlin

No incentive is provided under present-day specifications for the producer of concrete to use accurate control methods. The author recommends a straight strength specification, then it would be to the producer's advantage to have a scientifically designed mixture with the best water control and other control systems that he could get. Accurate batching equipment is needed and should be specified. Successively larger bonuses to the producer are suggested for each class of concrete within a limited strength range. If a minimum cement content is specified it should be based on approximate laboratory accuracy without factors of safety. If the cylinder tests are to govern extreme care is needed in the sampling, making, storing and testing of the specimens. The author believes that simpler specifications would result in a positive improvement in concrete quality and a reduction in its cost as well as reduction in the cost of inspection. The producer and consumer interests would be more parallel with a reduction in friction and evasion. The competitive position of concrete as a structural material would be improved.

Precise concrete curves in wind tunnel

Engineering News Record, V. 133, No. 6 (Aug. 10, 1944), pp. 108-110) Reviewed by S. J. Chamberlin

The 450-ft. long circuit varies in cross-section from 27 ft. 6 in. by 29 ft. 6 in. back of the fan to an 8 by 12 ft. test section with smooth changes of varying shape and length. The tunnel was built entirely in mass concrete to reduce vibration and improve operating conditions. Finished surfaces were brought to within $\frac{1}{16}$ in. of the designed dimension at the throat section and $\frac{5}{16}$ in. elsewhere without plaster finish of any kind. Most of the inner form faces were made with water-resistant plywood, carefully waxed and shellacked, and supported and stiffened to prevent movement. The inner form for the bellmouth was constructed on end with asbestos fiber plaster on expanded metal lath fastened to a wood backing. After the plaster was shaped to the right curvatures and hardened, it was waxed and shellacked and the whole form tilted into position. Part of the cage reinforcement was carried by overhead supports and part by concrete "chairs". The floor of the bellmouth, together with the side walls up to the level of the floor of the test section, was placed first as a separate casting, the balance being placed in a second operation. The only cold joints were horizontal, except at expansion joints where a steel slip plate was used, designed to function under either vacuum or pressure. Entrapped air in the wall forms was allowed to escape through $\frac{1}{4}$-in. holes bored through the forms. These holes did not leave form marks, evidently filling up after fulfilling their function.

Aircraft factory in France has concrete arch roof

Leon Beskin, *Civil Engineering*, V. 14, No. 6, June, 1944, pp. 240-242. Reviewed by J. R. Shank

An aircraft factory at Toulouse, France is an example of the ingenuity of French engineers where labor is cheap in comparison with materials. It also illustrates what can be done when advantage is taken by designers thoroughly conversant with mechanics, who take advantage of every method for reducing the quantities of materials used. The main building has a floor space clear of columns which is 170 x 660 ft. The con-

struction is completely of reinforced concrete. The amount of concrete used for the entire plant was equivalent to a 7 in. slab over the floor area. The steel used amounted to 9 lbs. But 3.5 sq. ft. of forms were necessary.

The plant consists of three main buildings and several extensions. On both sides of the main 170 x 660 ft. building are two-story bays of 50 ft. span. The roof of the main building is of saw-tooth construction on reinforced concrete trusses in the planes of the lights and mid-way between them, spanning 170 ft. The roof slabs are 2 in. sloped cylindrical shells spanning 16.5 ft. In addition to supporting the roof shells with all of their special thrusts the trusses support and are a part of the wind bracing and carry two run-ways at their third-points for three 55-ft. span, 3-ton traveling cranes. The 50-ft. span side buildings are designed to brace the main building in the planes of the trusses.

The article discusses the development of the composite design showing how the thin slab takes and delivers its loading. The concrete was designed for 4500 psi at 28 days. The steel had a minimum yield point of 50,000 psi and an ultimate strength of 80,000 psi.

Aircraft hangars and terminal buildings of reinforced concrete

CHARLES S. WHITNEY, *Aeronautical Engineering Review*, V. 3, No. 9, Sept. 1944.

From AUTHOR'S SYNOPSIS

In this time of rapid development of new materials in the aircraft industry, reinforced concrete should be given consideration because of its adaptability to the long spans that will be needed to house large aircraft.

Recent developments in the technique of the design and manufacture of concrete have been so rapid that its possibilities have not yet been fully developed.

Many things have been discovered in the last 10 years about the characteristics of concrete which permit more precise design of structures and more accurate control of the material. Transportation to the forms by pumping and consolidation by high-speed vibrators are examples of improved construction methods. Improvements in cement result in almost doubling the strength of concrete and increasing its reliability.

It is the purpose of this paper to point out what has been accomplished in the past and how concrete can be used in the construction of hangars and accessory space for any terminal facilities that may be needed.

The period immediately after the war will undoubtedly see a tremendous development of commercial airports. It is important that hangars and terminal buildings be substantially built and economically designed so as to result in the lowest possible operating cost to the air lines using them.

This paper describes a newly developed type of long-span reinforced concrete shell construction which should provide the kind of structure needed.

Concrete in Puerto Rico

H. K. EGGLESTON, *Civil Engineering*, V. 14, No. 6, June, 1944, pp. 237-239.

Reviewed by J. R. SHANK

The engineer inexperienced in concrete construction in the United States is surprised and disillusioned when he meets the materials available and the procedures common in Puerto Rico.

Three types of sands are available; beach, river, and crushed stone. The beach sands are of shell, coral, or silica, the silica very rare. Only one out of seven or eight beach sources is satisfactory. All beach sands are of one size, 30-50 or 50-100 mesh, rather fine but devoid of fines under 100 mesh. The structural qualities of the river sands are good, having been derived more or less from metamorphic and igneous rocks

but the grading is very coarse. The crushed stone sands are deficient in sizes below No. 16 mesh and contain excessive quantities of dust. These artificial sands are usually harsh and unworkable, requiring more cement per yard than natural sands, and give trouble in obtaining good surface finishes, as for floors. The best results are obtained by mixing the fine, uniform beach sands with the coarse river sands. Comparative tests with standard Ottawa sand mortars show the best of these sands to be about 75 per cent whereas 120 per cent is not uncommon for the glacial sands of northern United States. The beach sands contain some salt but this is not serious inasmuch as sea water is often used for mixing without an undue reduction in strength and durability.

Two types of cement are being produced in Puerto Rico. The Ponce cement is of the type having a so-called "moderate heat" of hydration or federal specification 206a. The Puerto Rico cement is a "standard portland" or federal specification 191b. The qualities of both compare favorably with similar types in the United States.

Field control is not always popular, however economical it may be. Such questionable practices as short mixing time, plastering, dry cement dusting, laying reinforcement on the bottom forms, poor construction jointing, and poor curing protection must be combatted.

Gouin Dam, Canada, repaired with Gunite

O. Graham, *Civil Engineering*, V. 14, No. 4, Apr. 1944, pp. 147-149. Reviewed by J. R. Shank

A power dam structure on the Upper St. Maurice River, 52 miles north of Sanmaur, Que., built in 1917-18, began to show scaling disintegration after 1930 which increased rapidly so that it became necessary to make repairs. The greatest deterioration occurred on the upstream face at the winter season water level. On the downstream face, the surface of the piers of the sluice section and at expansion joints showed advanced stages of deterioration, some defect as deep as 10 in.

The repairs, in order, consisted of unwatering the upstream face for a depth of about 20 ft., chipping out the deteriorated or loose concrete and cleaning the newly exposed surfaces, applying a metallic coating, fixing of special anchorages for wire mesh fabric, placing the wire mesh and guniting in the most approved manner.

Steel sheet-pile cofferdams were used on the upstream face where the depth did not exceed 25 ft. In deep water floating caissons with three walls and a bottom were made and equipped with air-tight buoyancy compartments, so designed that the amount of water and air in these compartments could be adjusted to float them to the face of the dam, and attach them by means of anchorages, previously prepared, to the concrete. Wood cushions were provided for contact with the concrete of the dam and divers calked them with wood wedges and canvas. After this the air and water in the compartments were adjusted so that they could be dewatered without producing undue stress on the anchorages.

The metallic coating was cast-iron powder and an oxidizing agent applied with a brush and allowed to oxidize. The wire mesh was No. 6 gage 3 in. x 3 in. mesh, anchored 20 in. on centers, staggered, placed within ½ in. of the concrete and 1¼ in. in from the finished surface of the gunite. The anchors were steel nails of a special make, ¼ in. diameter and 5 to 12 in. long with a two-unit expansion shield, two of lead and two of steel. A second layer of mesh was used when the thickness of the lining exceeded 3 in.

The gunite used was 3½ cu. ft. of sand, max. size ⅜ in. per bag of cement or four parts of sand to one of cement. The gunite was placed in layers not exceeding 2 in. at a time, each succeeding layer having begun to set before the next was begun. An admixture was used in the final layer on the upstream surfaces and the top of the dam to prevent setting cracks. The surface of the last layer was screeded and floated with wooden trowels.

Micromeritics: the technology of fine particles

J. M. DALLA VALLE, Pitman Publication Corp. (New York and Chicago), 1943. Price $8.50

Reviewed by D. E. PARSONS

This excellent book brings together for the first time a large amount of widely scattered theoretical and experimental information on methods of particle-measurement, size distributions, packings, dynamics of particles, physical properties of finely-divided substances, and related subjects, and its application, especially to problems in engineering, geology, hydrology and industry. The range of sizes of particles considered is limited to 10^{0} to 10^{5} microns, but this is a wide range as it extends from the upper limit of colloidal materials to the larger sizes of concrete coarse aggregates. Thus, the book considers most of the particle sizes that are dealt with by cement and concrete technologists and civil and mining engineers. However, the subject matter of the book is not limited to topics that are of concern only to a few professions or industries; the fundamental principles discussed and much of the empirical data are applicable to problems in many fields. Both theoretical and empirical equations are given when these are available, and each chapter concludes with problems illustrating the use of the equations

Chapter 1 discusses the scope and fields of application of the text. Chapter 2 begins with the application of dimensional analysis to problems relating to fine particles, using Buckingham's method, and then gives the various laws of motion of particles in a fluid. Chapter 3 is a discussion of the shape and size distribution of particles and contains an excellent summary and discussion of methods of representing particle shapes and sizes and particle-size distributions. Chapters 4 and 5 consider methods of particle-size measurement, giving detailed descriptions of direct and indirect methods of measurement. The discussion of sedimentation and sieving methods is especially comprehensive. Chapter 6 discusses such characteristics of packings as the size, shape and volume of voids, and the distribution of and effects of pressures. Chapter 7 discusses electrical, optical and sonic properties of fine particles, and Chapter 8 the thermodynamics of particles, including theoretical and experimental studies of heat effects and conduction, and adsorption, condensation and evaporation phenomena. Chapter 9 treats of such chemical properties as rates of solubility, crystal growth, and oxidation, and includes a brief discussion of dust explosions. Theoretical and experimental studies of the flow of fluids through packings are discussed comprehensively in Chapter 10 and percolation, moisture-holding capacity, infiltration and related subjects in Chapter 11. Chapter 12 considers capillarity. Chapter 13 gives methods for estimating the surface areas of particles from diameters and shape factors and by permeability, adsorption and optical methods. Chapter 14 on muds and slurries discusses such properties as viscosity, plasticity, consistency and settling. Chapter 15 includes an excellent summary of information on the transportation of particles by liquids and the movement of muds and slurries in pipes; Chapter 16 the theory of fine grinding and includes experimental data; Chapters 17 and 18 are of special interest to those concerned with the collection and separation of particles from air and the control of atmospheric and industrial dusts. A large and well selected bibliography is included and both the author and the subject indexes are excellently arranged.

The book covers the broad field of fine particle technology in a thorough manner. A very small amount of space only is used in presenting current specifications and empirical rules that are likely to be out of date soon; much of the material is fundamental and should require supplementing rather than radical revision during the next few years. Research workers and engineers interested in the crushing, grinding, grading and use of pulverized materials should find that the information given is a valuable supplement to the more specialized data in their respective fields. Some readers of the ACI JOURNAL may note the omission of discussions of dilatancy and thixotropy, prop-

erties that often are of interest and importance. The book includes an adequate mathematical treatment of the problems without an unnecessary use of higher mathematics, and interpretations that permit those not well versed in mathematics to apply the information without undue difficulty. The material is well arranged and is clearly and concisely presented; the book is well written. It should be useful to the student and as a handbook for specialists in all fields in which there are problems about the properties of finely-divided substances.

Expanding cements and their application—self-stressed concrete

A. Caquot and Henry Lossier, *Le Genie Civil*, V. CXXI, No. 8, Apr. 15, 1944, pp. 61-65 and No. 9 May 1, 1944 pp. 69-71. Reviewed by R. L. Bertin

In his introduction of Mr. Lossier's paper, Mr. A. Caquot describes briefly the principles and effects of prestressing concrete by embedding therein initially stressed reinforcing steel rods. Because of the quadruple effects of immediate elasticity, shrinkage, delayed elasticity and plasticity only a fraction of the initial induced compression remains eventually in the prestressed member. For that reason, steel of high yield value with large spring effect is most effective in ultimately maintaining a high percentage of the original compression. This concept was first applied to structures by Mr. Freyssinet.

Mr. Caquot cites the principle of hooping concrete conceived by Considere as another method of restrained concrete. In this case the transverse strains of the concrete are resisted by the transverse reinforcement, thus setting up a state of volumetric restraint giving to the concrete real ductility. Mr. Caquot introduces the method proposed by Mr. Henry Lossier whereby the reinforcing steel is pretensioned automatically by using in the concrete a cement possessing the special property of swelling regularly during the setting period. At the request of Mr. Lossier the research department of Poliet and Chausson, cement manufacturers, developed such a cement. This method is the subject matter of Mr. Lossier's paper.

The author presents a chronological review of the development of prestressed concrete, starting with the descriptions of German patents granted to Doehring in 1888 on prestressed concrete floor beams. United States patents to Hans in 1909 and to Crisenberry in 1915, both of which used prestressing of the reinforcing bars particularly for the purpose of holding them in exact position during construction.

He reviews tests conducted by Lund and Koenen in 1907 and by Considere in 1903 on prestressed reinforced concrete specimens. The originators of these tests had in mind the prevention of cracks in the tension zones of reinforced concrete structures.

The expanding cements are composed of three well known stable elements: (a) ordinary portland cement, (b) sulpho-aluminous cement which acts as the expanding factor, (c) a stabilizing element which after a regulated time stops the expansion by absorbing the principal reagent. The expansion both in intensity and duration can be regulated with surprising precision by judicious proportioning of the components. The mechanism of expansion from the chemical and physical standpoints is gone into in detail.

The physical characteristics derived from laboratory and field tests are described at length for four classifications of this special cement differing only in the magnitude of their total expansion of the neat paste cured under water as follows: (a) cement fictitiously called without shrinkage 0.2 to .3 per cent; (b) slightly expansive, .5 to .6 per cent; (c) moderately expansive .8 to 1.0 per cent and (d) strongly expansive, 1.2 to 1.5 per cent; these cements are intended for use in roads and pavements, arches and for voussoirs of arches and certain reinforced concrete elements.

The period of gradual expansion terminates in 10 to 15 days after mixing for submerged pure cement paste specimen. Curves showing the shrinkage and elongations of ordinary portland and the above listed cement specimens over a period of 5 years in terms of time both water cured and water and air cured indicate that those special cements cured under water expand gradually from .30 to 1.1 per cent in about 10 days when they become stabilized.

Those cured 7 days under water then in air exhibit about the same shrinkage characteristics from 7 days to 5 years as ordinary portland at which time they seem to stabilize with a residual expansion varying from .06 per cent for type (a) to .84 per cent for type (d).

The expansion of concrete made with these cements increases with the cement content up to 500 Kg per c.m. when it reaches about $\frac{3}{5}$ of that of neat cement after which the rate of increase drops very fast. The setting time is slower than that of portland cement requiring from 8 to 10 hours after mixing. The concrete compressive resistance generally is less than that of ordinary portland cement at early ages, about equal at 28 days and higher from then on. Concrete made with these cements is less permeable than that made with ordinary portland particularly if reinforced. Because of the high content of SO_3 the use of aggregate free of sulphates is mandatory.

Methods of testing concrete specimens made with these cements are described at length to establish the influence of composition and method of placing on the mechanical characteristics and the intensity, amplitude and energy of expansion.

A second series of tests is described relating to the adhesion between steel bars and expanding cement concrete. Also to the effect of moisture on thin and thick sections wetted on some and all of the faces. Full sized models were tested using the different classes of expanding cements in pavements, arches, combination structural steel and concrete floor systems, reinforced concrete beams and slabs.

Mr. Lossier sums up the result of his work as follows: Expanding cements have been studied and improved for more than eleven years and have reached a state of development which yields stable and well defined chemical and physical characteristics. The intensity and period of expansion are capable of exacting control both in the manufacture of the cement and in its application. The use of these cements does not differ from that of ordinary cement except for the necessity of water curing for the period of expansion—10 to 15 days. Eleven years of experimental research indicatex that concretes made with these cements are stable chemically and maintain their strength and acquired expansion. The compression resistance is at first less than that of portland cement but it gradually equals and ultimately surpasses it. When the expansion is restrained either by abutments or reinforcements, the gradual increase in compaction of the concrete increases its ultimate compressive resistance. The utilization of the energy of expansion of expanding cement concrete is found useful in the following types of structures:

Foundations—for all work cast against the earth as for instance, cast in place piles, footings, wells, etc. the resisted expansion of the concrete increases its compression and frictional resistance.

Underpinning—the expansion of the concrete takes the place of jacks in securing contact between the parts.

Repair of defective or accidently damaged work—wherever cavities or cracks in masonry structures need to be filled, the use of expanding mortar or concrete restores the structure in a manner impossible of attainment with ordinary cement.

Arched and bow string trusses—the use of expanding concrete permits the nullification of the action of shrinkage and normal plastic and elastic deformation by realizing an automatic decentering of the structure.

Tunnels, subterranian structures, dams and the like—grouting of such structures between the earth and their walls with expanding mortars improve their water tightness and eventual stability.

Pavements and roads—Observation of actual sections under working conditions indicates the possibility of eliminating the formation of cracks by suitably binding their boundaries.

Reinforced concrete—the energy of expansion of the concrete restrained by the reinforcing accomplishes automatically during the setting period, the work of prestressing utilized in structures such as arches, tanks, which can be extended advantageously to many other types such as bridges, ships, floor systems and the like.

23rd annual Proceedings of the Highway Research Board

HIGHWAY RESEARCH BOARD, 1944. Reviewed by FRED BURGGRAFF

The Proceedings of the Twenty-third Annual Meeting of the Highway Research Board contains 50 papers covering various phases of highway economics, design, materials, construction, maintenance, traffic, soils and aerial photography. Following are brief reviews of eleven of the papers, which contain information of interest to the concrete technician:

Report of Committee on Rigid Pavement Design, R. D. Bradbury, Chairman—The extensive program of airport construction during the past two years has centered attention more upon airports than upon highways. This has raised the question as to whether airport conditions are such as to require or permit substantial modification of the established principles of highway pavement design when applied to airports, notwithstanding the fact that both classes of pavement are subjected to the same kind of stress-producing conditions—namely, vehicular wheel loads and climatic exposure. Such differences in conditions as do exist are embodied largely in the relative magnitudes of applied wheel loads, their corresponding areas of distribution on the pavement surface, and possibly the relative frequencies of critical stress repetition. It is possible that the differences between airport and highway loadings—although differences in degree only—might be of such a high order of magnitude as to require a fundamental modification of the commonly-used theory of stress analysis.

In the highway field, certain information that is being disclosed by the service behavior of some of our pavements under wartime traffic may have an important influence on future design practice. In the case of trucks, more vehicles are using the highways and heavier loads are being carried. Whether due wholly to this over-loading or to a combination of causes, the fact is that some new concrete pavements built under wartime restrictions have, under wartime traffic, developed serious breakage after having been in service less than six months. Undoubtedly valuable lessons are to be learned from such cases.

Experiments with Continuous Reinforcement in Concrete Pavement—A Five-Year History by H. D. Cashell and S. W. Benham—In the fall of 1938 a number of experimental sections of reinforced concrete pavement were constructed near Stilesville, Ind., on U. S. Route 40 as a cooperative research project by the Public Roads Administration and the State Highway Commission of Indiana.

The data obtained during the 5-year period show that: (1) changes in pavement elevation have been generally small and non-uniform and there is nothing to indicate that these changes have affected the length changes and the crack patterns of the sections; (2) excepting the very short sections, daily and annual changes in section length are not directly proportional to length of section; (3) the magnitude of the restraint offered by the subgrade is a function of the time in which a given temperature

or moisture change occurs in the pavement; (4) maximum tensile stresses originating from subgrade restraint develop during the late summer and fall; (5) frequency of cracking increases with increase in section length; (6) surface appearance of cracks is a function of width and crack width decreases with increase in amount of longitudinal steel; (7) all of the cracks have remained so tightly closed that they have little if any structural significance.

Rigid Type Pavement Joints and Joint Spacing, by H. F. Clemmer—The Highway Department of the District of Columbia has found that the shrinkage of the concrete in setting will provide sufficient expansion space for pavements constructed during normal summer conditions, and that if planes of weakness are constructed at approximately 15 ft. intervals a certain degree of dowelling across the joints will be maintained.

Experiment and experience over ten years have proved that the division of concrete bases into slabs 12½ ft. long will provide definite control of cracking in sheet asphalt surfaces; the movement at any one joint being so slight as to be absorbed by the resilience of the bituminous surface.

The Department recommends the construction of reasonably short slabs, and when available the use of well distributed reinforcement, tie bars at all construction joints and load transfers across all transverse expansion and contraction joints.

Transverse Joints in the Design of Heavy Duty Concrete Pavements, by H. W. Giffin—This paper is an account of experience with concrete pavements in New Jersey. The situation of New Jersey is such that it is an ideal laboratory for quick and thorough testing designs of concrete pavements. Transverse joints, cracks, and pavement design are discussed as relating to ability to sustain the amount of heavy traffic using the State Highway System. Main conclusions reached are: (1) Heavy duty pavements require designs that provide load distribution at all joints and cracks; (2) Joint structures should be designed to reduce dowel restraint to the lowest practicable limit; (3) Surface water should be excluded from access to subgrade in so far as practicable; (4) The surface on which the pavement is laid should be non-erodible, have high bearing value, and preferably be somewhat porous; (5) Earth infiltration in open joints and cracks leads to spalling, blow-ups, and destruction of load transfer often accelerated by pavement growth; (6) Joint fillers should fill the joint space at all times; (7) The use of precompressed wood should be tried extensively for joint fillers; (8) Contraction joints are unsuitable for heavy duty concrete pavements and warping joints appear to be undesirable; (9) Concrete sills may be a better load distributing device than steel dowelled structures; (10) Consideration toward the early abandonment of the free end theory in construction of concrete pavements and adoption of the controlled compression theory, in which pavements are kept under compression at all times by the use of spring dowelled joint structures, is suggested as being more in accord with concrete's inherent characteristics.

Uncertainties in Design of Concrete Pavements Due to Differential Settlements and Volumetric Changes, by Francis M. Baron—The paper presents a qualitative and quantitative study of the effect of some variables on the design of concrete runways. Studied are the sensitivity of computed and measured strains, deflections, and subgrade reactions to uncertainties in variables of weather, differential settlements, and properties of materials.

Strains caused by volumetric changes of the material of the pavement and of subgrade may be, and often are, of primary importance. In some regions the design of a runway may depend upon designing for weather rather than for present loads of air carriers. Strains caused by wheel loads, however, are of interest as they may affect progressive demoralization of the runway. Clearly the importance of each source of strain

depends on the scale, relative magnitude, range of uncertainty and also on the character of the source. Design requirements of runway and subgrade for purposes of weather may be opposite to those for purposes of load.

Geometry of the deflected pavement is the dominating characteristic of deformations produced by volumetric changes of subgrade or of the material of the pavement. Strains produced by these sources are studied essentially as problems of geometry and not of stress.

Temperature Changes and Duration of High and Low Temperatures in a Concrete Pavement, by W. J. Arndt—The data for this paper were obtained by the State Highway Commission of Kansas during 1936 to 1941, as a part of an experimental concrete pavement project having various types of subgrade concrete and expansion joints, under observation in Douglas County, Kan. The temperatures observed were continuous during the period of observation. A summary of the extreme high temperatures and the extreme low temperatures recorded 1 in. from the top of the pavement and 1 in. from the bottom of the pavement and from the subgrade is presented.

Considerable significance is attached to the amount and duration of the high temperatures recorded, and to the amount and duration of the low temperatures.

(1) The low temperature data are significant with regard to a freeze and thaw soundness test both as to the temperature to be used and the number of cycles to be considered.

(2) No reliance can be placed upon a study of air temperatures when one is considering concrete pavement temperatures. Naturally as the air temperatures become higher the pavement temperatures will also become higher but they do not follow any given formula or any set pattern.

(3) Any study of temperatures should be for a considerable period. No one of the years during which these data were recorded was above or below average normal temperatures by more than about 2 degrees F. Yet, any one year of the five would present a far different picture from the others.

Progress Report on California Experience with Cement Treated Bases, by T. E. Stanton, F. N. Hveem and J. L. Beatty—The California Division of Highways has built 123 miles of pavement base by mixing cement with granular materials of many kinds and compacting on the subgrade by rolling or tamping. Most of these bases have been for first class road improvements. The materials have included fine silty sands, streambed gravels, disintegrated granite, soft crushed sandstone, fairly clean sand and aggregates suitable for concrete, and many construction methods have been tried. Twenty-eight projects ranging in length from one-half mile to 13.1 miles have been constructed.

Since in most cases the material for cement treated bases in California is brought from an approved pit or quarry it was found economical and otherwise desirable to use plant mixing rather than road mixing.

These cement treated bases are giving satisfactory service. Attempts to use thin bituminous surfaces on the cement treated bases did not give satisfactory results and hence a 3-in. layer of bituminous surfacing has been adopted as standard practice on California primary roads of moderate to heavy traffic.

The mixture design is based on compressive strength requirements of 850 psi at 7 days and 1000 psi at 28 days.

Effect of Calcium Chloride on the Water Requirements, Specific Weights and Compressive Strengths of Concretes Made with Plain and Treated Cements, by H. C. Vollmer—Slump

and flow tests were made on concretes using a portland cement, a wide range of water-cement ratios, and two cement factors. Comparative tests were made on concretes containing two per cent commercial calcium chloride. Calcium chloride increased the workability of all concretes tested, being slightly more effective on the richer mix in the range of slump of two to six inches.

Water requirement, compressive strength and specific weight tests were also made on concretes of constant slump using four plain cements and four cements containing an interground air-entraining agent. Comparative tests were made with two per cent calcium chloride. Compared to the plain concretes, the concretes made with treated cements required from 0 to 2 per cent less mixing water and had lower specific weights, and in most cases had lower compressive strengths; the addition of calcium chloride to the vinsol resin cement concretes decreased the required water and increased the early strengths and the specific weights.

Pumping of Rigid Pavements in Indiana, by K. B. Woods and T. E. Shelburne—This paper is a report of one of the researches conducted by the Highway Research Laboratories of Purdue University co-operating with the State Highway Commission of Indiana. Pumping action as well as the factors affecting pumping are described. Surveys of Indiana pavements show that approximately 250 miles (6 per cent) of the concrete pavements are subject to this action. In addition many hundred miles of pumping pavements have been observed in other Midwestern, Southern, and Southwestern states

Pumping is prevalent during periods of heavy rainfall on those roads which carry a large volume of heavy truck traffic. However, portions only of these roads, where certain soils predominate were found to be pumping. Test results on samples of pumping soils show that pumping generally occurs on plastic clay-like materials. Observations and results of tests on soils in several states outside of Indiana where pumping was found likewise show plastic clay-like materials. In Indiana these soil areas are lacustrine deposits, unweathered parent materials of the Wisconsin drift, weathered shales, and soils with claypan or "B" horizon development.

These studies reveal that on new construction, pumping can be minimized or eliminated by employing corrective measures. Drained insulation courses are particularly effective where plastic soils are encountered. Because of the magnitude of the problem such design features as expansion joints and load transfer devices should receive added attention. The maintenance or salvaging of the several hundred miles of pumping pavements is a serious problem, without an entirely satisfactory solution, confronting highway engineers during this critical war period.

Deflectometer for Meausuring Concrete Pavement Deflections Under Moving Loads, by R. W. Couch—The deflectometer was designed to measure concrete pavement deflections at any desired point under moving loads. The instrument that has been developed consists essentially of a ratio arm of 25 to 1 obtained by means of a train of gears and of a Kymograph drum on which deflections, speed-switch indicators and a time-graph are recorded simultaneously. Deflections of a slab at the point to be measured may be determined for any position of the moving axle load by calculating distances from the average speed of the moving load. Timing is controlled by a standard tuning fork of 100 vibrations per second. With an optimum speed of the kymograph drum, it is possible to determine the time within plus or minus 0.001 sec. Preliminary tests by static displacements show that it is possible to measure displacements of the magnification arm within plus or minus 0.001 in. Actual displacements are measured with an "88" Ames

dial and magnified displacements on the kymograph chart are measured with a pair of microcalipers.

Effect of Soil and Calcium Chloride Admixtures on Soil-Cement Mixtures, by M. D. Catton and E. J. Felt—This report shows that some sandy surface soils which react poorly with cement, and therefore require high cement contents for hardening, can be improved to react in the normal manner by adding to the sand an admixture of clayey soil, or by adding a small quantity of calcium chloride.

Compressive strength, wet-dry and freeze-thaw test data are given showing the effect of the soil and clacium chloride admixtures on a number of poorly reacting sandy soils. Data are also given showing the effect of calcium chloride upon normally well reacting soils.

Construction costs are analyzed for poorly reacting sandy soils with indications that the costs may be excessive when cement alone is used. However, the admixture of clayey soil or calcium chloride to these soils, with accompanying reductions in cement requirements, will often result in costs similar to those prevailing on projects where these special problems are not present.

ACI NEWS LETTER

Vol. 16 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1945

1945 CONVENTION MUCH CURTAILED

Business and Administrative Sessions Only

It is with very deep regret that the Board of Direction announces the cancellation of most of the plans for the 41st annual ACI convention as announced for February 13 to 16, 1945, Hotel New Yorker, New York City.

The decision was made with a high degree of Board unanimity on the basis of an urgent request from the Office of Defense Transportation. O. D. T. requests are not mandatory—many meetings are being held more or less "as usual", but the Board action made it clear that the Institute wishes to cooperate as fully as possible with the war effort, even at considerable sacrifice of its own interests in the convention's stimulation of its efforts in a specialized public service.

The 41st annual convention will include none of the customary technical sessions. Only such meetings are now scheduled as seem necessary to effective administration and the "legal formalities" of administrative succession under our by-laws; anticipated travel reduced by 90 percent.

Meetings will be as follows:

Board of Direction meets 12 o'clock, noon, at lunch Wednesday, February 14 with sessions continuing through dinner into the evening.

Advisory Committee, Raymond E. Davis, Chairman, meets 10 a.m. Thursday, February 15 and continues through the afternoon.

Publications Committee, Douglas E. Parsons, Chairman, meets 6:00 p.m. at dinner, Thursday February 15, through the evening and continuing at 9 a.m. Friday, February 16 until 11:45 a.m.

General session of ACI (the 1945 Convention) beginning at luncheon 12 o'clock noon, Friday February 16. Following the coffee: the report of tellers and induction of new officers and directors; an address by the retiring president; the award of Wason Medals (see elsewhere in these pages) and matters pertinent to the administration of Institute affairs.

To ensure the necessary quorum at the general session, dependence will be on New York City Members plus the members of the Board and of the Advisory and the Publications Committees. Local, New York members are especially urged to attend. Special provision will be made for advance reservations for the luncheon. Similar opportunity is of course open to any Institute members who plan to be in New York at that time. To observe the spirit of the compliance with the O. D. T. request (especially as no technical reports or papers will be presented for discussion) distant members are not urged to attend, except as they are included in the personnel of the meetings scheduled or are, presumptively, members-elect of the Board of Direction. (Since the annual ballot involves no contests for Board membership, the election of candidates might be assumed.)

A brief meeting of the "new Board" is scheduled to follow immediately after the general session.

The 42nd annual Convention is scheduled for New York City, February 18 to 21, 1946.

Wason Medals Won by Commodore Angas, Lt. Commander Shanley, Lieutenant Erickson and Harrison F. Gonnerman

Annual awards of Wason Medals are announced by the Board of Direction:

For "The Most Meritorious Paper" in ACI Proceedings V. 40, Commodore W. Mack Angas (CEC) USN, Lt. Comdr. E. M. Shanley (CEC) USNR, and Lt. John A. Erickson (CEC) USN, "Concrete Problems in the Construction Graving Docks by the Tremie Method" published ACI JOURNAL February 1944. This is the third successive award of this medal to members of the Civil Engineers Corps., U. S. Navy (for his 1942 paper "Architectural Concrete in the New Naval Medical Center" to Capt. Hugo C. Fischer; for their 1943 paper "The Properties and Behavior Underwater of Plastic Concrete" to Capt. P. J. Halloran and Kenneth H. Talbot.)

For "Noteworthy Research" reported in ACI Proceedings V. 40, to Harrison F. Gonnerman in the work reported in his paper "Tests of Concrete Containing Air-entraining Portland Cements or Air-entraining Materials added to Batch at Mixer," published ACI JOURNAL, June 1944. This will be Mr. Gonnerman's second research medal, he, with Paul M. Woodworth, having won it for the work reported in their 1929 paper "Tests of Retempered Concrete".

The Wason medals and certificates of award will have formal presentation at the luncheon meeting of the American Concrete Institute, Hotel New Yorker, New York City, at noon Friday, February 16th at which the medal winners will be Institute guests of honor.

New Members

The Board of Direction approved 37 applications for membership received in October and November (28 Individuals—1 Contributing—2 Corporation, 5 Junior and 1 Student) as follows:

Bamfield, A. E., Australian Cement, Ltd., Geelong, Victoria, Australia

Berg, Oswald, Jr., 1804 Lee Highway, Arlington, Va.

Blake, Leslie C., 50 Dyer Ave., Milton, Mass.

Boland, Fred L., 511 Jackson St., Petoskey, Mich.

Borden, R. C., 211 South Farr St., Provo, Utah

Brewer, Harold W., Bureau of Reclamation, Denver, Colo.

Chisholm, D. A., Dept. of Highway & Public Works, Province Bldg., Halifax, N. S., Canada

Cococcio, V. J., CMlc, B-3 14th USNCB, c/o Fleet Post Office, San Francisco, Calif.

Douglas, A. H., 65 Crichton St., Ottawa, Ontario

Gindi, Adly W., 20, Memary Pachu St., Sakaking, Cairo, Egypt

Glaze, V. L., 1402 51st St., Sacramento, Calif.

Guest, J. E., 49 North Lane, Astley, Manchester, England

Howe, Myron A., 133 Hawthorne Ave., Needham 92, Mass.

Jackman, H., Mt. Wellington Highway, Auckland, S. E. 6., New Zealand

Keniston, T. Sgt., Edward F., 3532 Center St. N. W., Washington 10, D. C.

*KOEHRING CO., 3026 West Concordia Ave., Milwaukee 10, Wisc. (R. A. Beckwith)

Laboratory for Testing Materials, 200 Dizengoff St., Tel-Aviv, Palestine (A. Arnstein)

Mauchel, Robert L., c/o Master Builders Co., 101 Park Ave., New York 17, N. Y.

McClelland, James E., 2840 Madison Rd., Cincinnati 9, Ohio

McDonell, Hubert F., 438 S. Arredonde St., Gainesville, Fla.

McGrail, Paul J., 1943 Semple St., St. Louis, Mo.

Misavage, B. A., 31 S. Locust St., Mt. Carmel, Pa.

Montgomery, Richard T., P. O. Box 844, Estes Park, Colo.

Nietz, Ernest W., P. O. Box No. 97, Gastonia, N. C.

Pennsylvania Commonwealth of, Dept. of Property & Supplies, Harrisburg, Pa. (A. Judson Warlow, Chief Engr.)

Pirtz, David, 2575 LeConte Ave., Berkeley, Calif.

Skou, C. Chr., Gimle, The Grove, Marton-in-Cleveland, Middlesbrough, Yorks, England

Sleath, Aubrey B., Box 533 N., Niantic, Conn.

Stewart, Leslie M., 155 Highland Ave., Winchester, Mass.

Sussman, William, 649 Park Ave., East Orange, N. J.

Tosca, Ernesto, Sta. Catalina No. 212 Rpto., Mendoza, Vibora, Havana, Cuba

Van Der Mark, H. J. C., P. O. Box 5467, Johannesburg, Transvaal, So. Africa (Maas & Van Der Mark)

Whelan, Donald V., 207 E. St. N. W., Washington 1, D. C.

Winslow, Ralph E., Dept. of Architecture, Rensselaer Polytechnic Institute, Troy, N. Y.

Witte, Leslie P., 1585 Elm St., Denver 7, Colo.

Wolff, Chas. R., Jr., 113 Third Ave., Columbia, Tenn.

Woodhouse, Earl H., 4095 Albatross, San Diego 3, Calif.

WHO'S WHO

Lewis H. Tuthill

has been an ACI member since 1926 and for most of that time an active one. His present paper (p. 137) is not only an important record of construction practice in World War II's concrete shipbuilding but is so presented as to involve a wealth of information applicable to high quality concrete construction for any purpose.

Mr. Tuthill is not by any means new to ACI JOURNAL pages as the record on page 96 of the current ACI Directory shows. He was chairman of Committee 614, whose report was adopted 1942, "Standard Recommended Practice for Measuring, Mixing and Placing Concrete". He has been an active worker for new ACI members as Honor Roll records will show. He was long with the Metropolitan Water District of Southern California and since 1939 with the Bureau of Reclamation, except for his recent work on concrete ships; from November 1942 to March 1943 with McCloskey & Co., Tampa, Fla. and then with the United States Maritime Commission until July, 1944 when he returned to duty with the Bureau of Reclamation. His contributions to and wide knowledge of concrete construction practice led to his appointment to the Advisory Committee recently as Chairman of Department 600, Construction.

Paul William Abeles

ACI Member since 1941, was born at Mistelbach, near Vienna, Austria, Jan. 17, 1897. He attended Classical High School, Vienna, and was graduated from Technical University, Vienna (Austrian Eng. Diploma after 5 years' course in Civil Engineering) 1921. In 1928 he took his degree of doctor of engineering (Dr. Ing.).

1922 to 1923 he worked as structural engineer with the Building Authorities at Recklinghausen, German Ruhr-coal district (examination of statical calculations and supervision of erection of industrial buildings for 17 coal mines); 1922-29 was first designer and later head of the designing and estimating department of N. Rolla & Neffe, (building and civil engineering contractors), Vienna; in this capacity designed the frame constructions of the Amalia bath, Vienna. From 1929-1939 he was a consulting engineer in Vienna, mainly engaged in industrial buildings in Central and Southeastern Europe. He carried out extensive tests on spun concrete poles, tubular and half-tubular beams and on concrete reinforced with high strength steel, with a special view to the behavior at cracking and at failure, and to the danger of rusting.

He left Austria in 1939 for London as refugee from Nazi oppression; worked in London first as designer, then as technical advisor and consultant in structural and civil engineering and investigated "prestressing".

He published simplified methods on the calculations of frame structures by the four moment theorem in *Beton und Eisen* (1924 and 1929) and in *Concrete & Constructional Engineers* (1942). A textbook on "Statics of Building Construction" appeared in Vienna 1931. Reports on the tests on spun concrete and on high strength reinforced concrete as well as treatises on the danger of rusting, on high strength reinforced concrete and on the elasticity of concrete were published in *Beton und Eisen* (1935 and 1937). *Zement* (1937) and *Concrete and Constructional Engineering* (1940 and 1942). Reports on the practical application of spun concrete, poles and floors appeared in *Zeitschrift d. Oe.I. & A.V.* (1935 and 1937). Various other publications appeared in periodicals and books. He has been a contributor to discussion in the ACI JOURNAL but this issue carries his first paper (p. 181).

Bartlett G. Long, Henry J. Kurtz and Thomas Sandenaw

who make their first appearance as authors of a paper in this JOURNAL (p. 217) are all ACI members—Long since 1934, Kurtz since 1942, Sandenaw since 1943, all of them until recently actively identified with the Army Engineer Corps' Cincinnati Testing Laboratory, at Mariemont, Ohio.

"Bart" Long, well known to many ACI members, severed his connection with the laboratory last summer and joined Mrs. Long on their ranch at Pecos, N. M. The last word ACI had from him, expressing an interest in seeing proofs of the paper, was a letter written from a Santa Fe hospital where he was getting on his feet again after being struck down by coronary thrombosis.

Mr. Long was born in St. Louis, Missouri, Apr. 3, 1892. He attended the University of Missouri, from Sept. 1911 to June, 1912, and the Washington University, from Sept. 1912 to June, 1914. He was a 1st Lt. in World War I from July 26, 1917 to May 2, 1919, having served as an instructor in the Air Service.

His professional experience includes employment by Nelson Cunliff, Engineer, and Park Commission, St. Louis, Missouri; Florida State Road Dept., Tallahassee; Woods-Hoskins-Young Co., Chicago; State Highway Department, Santa Fe, New Mexico; Bureau of Reclamation, Denver, Colo.; War Department, U. S. Engineers, Seattle; U. S. National Park Service, San Francisco; War Department, U. S. Engineers, St. Louis, and Mariemont,Ohio.

Mr. Kurtz was born in Flint, Mich., Jan. 8, 1897; was graduated from Michi-

gan State College, B.S. in 1921, M.S. in 1928.

His professional experience includes employment by Consumers Power Co., Battle Creek, Mich.; Commonwealth Power Corp., Jackson, Mich.; Pacific Telephone & Telegraph Co., Sacramento, Calif.; Bell Telephone Laboratories, New York City; Remler Radio Co., Inc., San Francisco; National Bureau of Standards, Washington, D. C.; American Society for Testing Materials, Washington, D. C.; U. S. Coast Guard, Cambridge, Mass.; War Department, U. S. Engineers, Mariemont, Ohio.

Mr. Sandenaw took his B. S. in industrial chemistry from Montana State College in 1934; attended Ohio State University for graduate work in chemistry in 1935-36; was employed in the chemistry section United States Engineers laboratory Fort Peck, Montana; 1936-38, took an M. S. degree in chemistry from Carnegie Institute of Technology in 1939; analytical chemist in the U. S. Navy Inspection laboratory Marshall, Penna., 1939-41; employed in the chemical section of General Materials Laboratory, special engineering division, the Panama Canal, Diablo Heights, C. Z. 1941-43; in the concrete section Cincinnati Testing Laboratory, Mariemont, Ohio, 1943-44. He is now employed by the RCA Victor Division of Radio Corp. of America as a chemist in the engineering department of the vacuum tube manufacturing plant.

Harry F. Thomson

vice president, General Material Co., St. Louis, Mo. a member of the Institute since 1928, and on two occasions the author of a paper in the ACI Proceedings, a member of the ACI Board of Direction 1939 and 1940 as director for the Fifth district, again as director-at-large since 1942 has been nominated by the American Society of Civil Engineers as director for the ASCE district fourteen.

Alfred E. Lindau

died at Pearl Harbor, Hawaii, December 14. He had completed five years service as Chief Civilian Engineering Assistant to the Officer-in-Charge, Pacific Naval Air Bases—with a range of engineering service and responsibility exceeding the implications of his official title. He was reported to have been in good health; "suffered no illness; death was sudden from a heart attack."*

Very few, if any, ACI members had contributed so much over so many years and from such a wide range of activity as had Alfred Lindau and few indeed have been held in such esteem by their fellow members—for a high degree of intellectual integrity, a broad cultural background, a lively interest in a wide range of human experience and a deep human understanding. He had the rare combination of objective judicial-mindedness with great warmth of personality. His kindness was unfailing. He was unexcelled as an Institute convention presiding officer. He brought to the leadership of discussion not only his broad knowledge but his dynamic spirit and a contageous interest.

Lindau had been a member of ACI since 1909—an Honorary Member since 1935. He was a member of the Board as a Director 1915 and 1916 and continuously a Board member from 1922-1935—as vice-president 1922 and 1923, president 1924 and 1925, past president member 1926 to 1935.

Lindau was born in Sweden, March 15, 1874. An only child, and his father having died, he was brought to this country by

*In a personal letter written December 10, four days before he died (but received by the ACI Secretary more than 2 weeks later), Lindau had said " . . . I'm just now taking your advice to slow down. The pace has been a little fast for the last two or three years. I feel that the peak of construction has passed and that the facilities for carrying on the war have about been completed . . . So-called permanent construction has given way to temporary buildings expected to last out the war." He said he had recently taken up two engineering subjects, soil mechanics and hydraulics of water distribution and had been supervising the testing of runway slabs for wheel loads far in excess of those reported thus far. "I may have something to send you one of these days when war censorship permits free elbow room."

his mother at the age of eight. He went through grade school in Chicago and worked in engineering offices as office boy, later getting some high school work, but did not complete a regular course. Admitted to the University of Michigan with much less than the usual preparatory work, he was graduated class of 1900. He won the interest of several professors who helped him acquire that foundation of mathematical and scientific knowledge on which he so richly capitalized. Some of his early engineering work was with the C. B. and Q. and the Rock-island railways. He was assistant bridge engineer with the Rock Island when he quit railroading to join A. L. Johnson, inventor of the corrugated bar in the Corrugated Bar Co., of which Lindau became chief engineer, then sales manager, vice-president. When Kalman Steel Co. took over Corrugated Bar (1924), Lindau became president of American System of Reinforcing with a plant at Libertyville, Ill. This enterprise was wiped out so far as Lindau was concerned by the depression years. With an agency for Viber concrete vibrators he entered upon a period of intensive work in their demonstration; was struck down by coronary thrombosis. He made such a complete recovery of his health that he later passed a rigorous examination for civil engineering consulting service with the Navy. Writing a friend from Pearl Harbor soon after Dec. 7, 1941, he said "the longest day I ever lived."

When Rear Admiral Ben Moreell (now Vice-Admiral), became Chief of the Navy's Bureau of Yards and Docks late in 1937, his first concern was to expand the shore facilities of the Navy as rapidly as possible to be ready for whatever might happen. Funds became available in June 1939 and the first contract to be awarded (August 5, 1939) was for the construction of the Pacific Naval Air Bases at a cost of $15 million—later expanded progressively to more than $300 million. The contract was also awarded for dry docks at a total of $18 million.

Alfred E. Lindau

The new construction involved important use of reinforced concrete in many diverse and novel applications, ranging from huge battleship graving docks for which structural concrete was placed under sixty feet of water, to batteries of concrete oil storage reservoirs built in the heart of a mountain. Alfred Lindau was particularly well qualified to assist and advise the officers of the Civil Engineer Corps of the Navy on the design and construction of such projects and was so engaged from the date of his arrival at Pearl Harbor, December 1, 1939 to the day of his death. The excellence of his work at all times was recognized, and his status advanced in July 1942. Due to the unusual type and characteristics of aggregates available in the Hawaiian Islands and to the use of tremies to place structural concrete in deep sea water, it was necessary to carry out extensive research in the design of a satisfactory mix that could be placed by the proposed means. Tests included the casting of large blocks of concrete (weighing up to 134 tons) in deep water, which were raised and examined. Twenty-two

such blocks were made to assure fully satisfactory results in construction. Similar problems were successfully met in connection with special concrete work on other projects and with unusual soil conditions at many places. Dredged coral for concrete aggregate was tested for use on other islands. Lindau's work on materials, design and superintendence of construction included projects at Midway and other outlying islands—projects of supreme importance in the conduct of the war in the Pacific. His intellectual honesty, his long experience, and great human kindness and understanding won for him the respect and affection of all the officers and civilians with whom he worked. Characteristically, he aligned himself immediately with the Hawaiian Chapter of the American Society of Civil Engineers, presented and discussed papers at their meetings, served as vice president in 1942 and having been elected president was to have been installed in that office January, 1945.

Mr. Lindau's services to the Institute are all too briefly set down in the record following his name, page 68 of the 1944 ACI Directory beginning with his first paper in 1910. He worked on many committees; he had been a member of the first Joint Committee on concrete and reinforced concrete which reported in 1916, the second Joint Committee, 1920 to 1924; he became chairman of the third Joint Committee in 1932, following the death of Willis A. Slater; continued to head the committee's work until completed with the publication of the current report in 1940.

Mrs. Lindau was with her husband at Pearl Harbor. He is also survived by two daughters, Mrs. Allison O. Hunt, of Wooster, Ohio, and Capt. Marjorie Lindau, Army Nursing Corps., somewhere in France, and grandson, Stephen Hunt.

It seems appropriate to republish a sketch of Mr. Lindau and his activities written by Arthur R. Lord. It was one of a series of brief essays in which Mr. Lord discussed "Fellows of the Institute." Lindau's activities in the thirteen and a half intervening years have served to underscore much of what Mr. Lord wrote then.

FELLOWS OF THE INSTITUTE*—ALFRED E. LINDAU

BY ARTHUR R. LORD

(from *News Letter*, ACI Journal, Sept. 1931)

What is the secret formula for eternal youth that this man has somehow discovered when so many others have failed dismally in the quest? We can look back on the beginnings of reinforced concrete and there Lindau stands calmly doing a liberal share of the pioneer designing, writing the early papers and discussions, along with Talbot, Hatt and Turneaure. We can see him now no less, still active in the Institute's affairs, still leading at the thin edge of progressive advance. If he were not so masterfully a part of the present and the future we would expect to find him lonesome for many of his companions of the past. But he is a difficult subject to imagine as solitary or lonesome in any age.

And the ranging sweep of his enthusiasms! Are you fond of good books, ancient classics, modern rebellions? Alfred has read them, will be only too happy to discuss them. Are gardens and hills and natural beauty your sources of strength and peace? They are his, no less. Do you find joy in walloping an inoffensive ball as far and as accurately as you can? He's up at the crack of day (with John Ahlers) to get in one more round than the rest of us consider necessary for a golfful day. Is your recreation aided by mathematical analysis, mysterious Greek symbols and page-long equations? Well, Alfred shows every indication of heartily enjoying the conversation of Westergaard and Timoshenko when they are whooping up their specialty. And of course we all know that he sells steel and service as a bread-and-

*A special order of distinction to which nominations are made from among his friends and associates in the Institute work by the author of a series of biographical sketches of which this is the fifth—Editor.

butter diversion through his American System of Reinforcing.

Identified actively and prominently—almost I had said exclusively—with concrete, Alfred E. Lindau has happily avoided the professional bigotry, the narrowness of many specialists. He's an acceptable and invited speaker before the American Institute of Steel Construction. He presides over meetings at which wood and steel and concrete lie down together as lambs in the fold. Reaching out eagerly for those original contributions of thought and experience that others offer, he does not fail to repay his debts in kind and in handsome measure. He has served the American Concrete Institute with long maintained distinction, travelling great distances, displaying unflagging interest and enthusiasm, as Committeeman and as President.

In such a full rounded life it would be useless to emphasize any single achievement. Some of us hammer away so long and so noisily, driving our special spike in the Palace of Progress, that the calcium light is turned on us finally to discover what in the world all the disturbance can be about. Alfred travels so extensively and so rapidly over the entire structure, from foundations to roof (and over the whole neighborhood for that matter), that nothing less than a general illumination would serve to disclose at all adequately, to appreciate at all properly, his contributions. Each department proves too fascinating to him to permit any other to monopolize his gallant attentions. Not connected with any college faculty, he yet possesses to a superlative degree those qualities that have made certain professors preeminently useful—sympathy, understanding, outlook, inspiration, personality, humor. Some university has lost—a far greater world has gained—a wise counsellor and friendly leader by the accident that Lindau, out of many absorbing interests, chose business as the objective of his directed hours. He would have been successful likewise in any one of several engineering professions.

Frank William Capp

who for 17 years as a structural engineer with the Portland Cement Association had devoted most of his time to concrete problems of the railroad field, died of a heart attack the morning of December 22, four minutes after reaching the PCA offices, apparently in perfect health. He was not an ACI member but had contributed to its work, notably at the 1936 convention with a paper "Maintaining Concrete Structures" (ACI JOURNAL, May-June 1936).

He was born in St. John, New Brunswick, Canada, June 6, 1887, the son of a minister, Thomas Henry and Ada Ella (Baker) Capp. He was brought to the United States by his parents in 1891, and spent most of his boyhood and early manhood in Missouri. Throughout this period Mr. Capp read much in his father's library which, together with his early training imparted by his father, influenced his life greatly. He was graduated from the University of Missouri in 1909 with the degree of Bachelor of Science in Civil Engineering. He was awarded the degree of Civil Engineer by his Alma Mater in 1912. After his first work as a draftsman with the American Bridge Co. in Toledo, from 1909 to 1910 he became instructor in Civil Engineering at the University of Missouri, leaving there in 1912 to get into engineering practice with the Kansas City Terminal Railway for a short period and then with the bridge department of the Great Northern Railway, St. Paul, Minn., where, as a designer he remained from 1913 to 1920. He then joined the staff of Adolph F. Meyer, consulting hydraulics engineer in Minneapolis as an assistant engineer during the period from 1920 to 1924. He got back into railroad work in 1924 as an assistant engineer on the Chicago terminal improvements of the Illinois Central Railroad, leaving that work in 1927 when he joined the staff of the Portland Cement Association as structural engineer where he was em-

ployed chiefly on railway problems up to the time of his death.

He was active on the Committee on Masonry and the Committee on Roadway and Ballast of the American Railway Engineering Association and on a technical committee of the Way and Structures Division of the American Transit Engineering Association to which he contributed generously of his time—a member of both organizations, Mr. Capp's intimate knowledge of concrete and of railroad work made his counsel on these committees invaluable.

Mr. Capp made a careful study of railroad track construction and maintenance problems. In that connection he conceived the possibility of stabilizing track and thereby reducing maintenance cost as well as improving the riding quality of the track by injecting portland cement grout into the subgrade under pressure. The first work of this nature was done under Mr. Capp's supervision in 1936 and since that time the practice of pressure grouting roadbed has been adopted by most of the major railroads of this country

June 8, 1917 Mr. Capp married Kitcy Helen Roberts of Chicago. Mrs. Capp and his two children, Thomas Milton and Cornelia Helen survive him. The son was home on furlough after two and a half years service in the Army, mostly in North Africa and at Foggia Field, Italy. The family was planning what promised to be its most delightful Christmas day since the son and daughter were very young.

Frank Capp had a host of friends and was held in high esteem in the railway field, where he had done much valuable work, and elsewhere.

If you are going to be in New York City, February 16 and wish to attend the one brief ACI Convention Luncheon Session, write the ACI Secretary and buy a luncheon ticket—$2.25. All "noses" must be counted in advance.

Honor Roll

Feb. 1 through December 31, 1944

ACI Members listed have proposed new ACI Members as shown after their names. There should be more names on this list and more new members tallied. It is all a matter of being "Member Conscious".

F. E. Richart	13
Mehmet Tokay	8
Lewis H. Tuthill	8
J. W. Kelly	6
A. J. Boase	5½
Jacob J. Creskoff	5
Harry Erps	4
William A. Jones	4
Charles E. Wuerpel	3½
J. A. Crofts	3
Kenneth K. Knight	3
F. W. Panhorst	3
H. F. Gonnerman	2½
Douglas E. Parsons	2½
Hugh Barnes	2
Elmer B. Belt	2
R. F. Blanks	2
C. E. Everhart	2
P. J. Freeman	2
Ernst Gruenwald	2
Fred E. Hale	2
W. G. McFarland	2
H. W. Mundt	2
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Kenneth H. Talbot	2
A. E. Cashe	1½
Raymond E. Davis	1½
H. B. Emerson	1½
Bengt. Friberg	1½
Albert Haertlein	1½
Warren Raeder	1½
K. E. Whitman	1½
R. R. Zipprodt	1½
Owen Arthur Aisher	1
R. Howard Annin	1
L. J. Baistow	1
J. F. Barton	1
S. D. Burks	1
Morgan R. Butler	1
Julian B. Carson	1
S. J. Chamberlin	1
H. Clare	1

H. F. Clemmer....1
Joseph Di Stasio....1
B. M. Dornblatt....1
John J. Earley....1
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H. D. Farmer....1
H. E. French....1
H. M. Hadley....1
Elliot A. Haller....1
G. H. Hodgson....1
W. M. Honour....1
Norman J. Huber....1
M. E. James....1
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Paul F. Keatinge....1
Henry L. Kennedy....1
W. E. Lumb....1
Donald R. MacPherson....1
H. E. A. McCarty....1
Roscoe J. Mason....1
Rene Paulido Morales....1
Ben E. Nutter....1
R. A. Plumb....1
John W. Poulter....1
Walter H. Price....1
Ansel T. Rogers....1
John R. Ruhling....1
M. K. Scheirer....1
Ralph A. Sherman....1
R. T. Sherrod....1
B. Skramtaiev....1
Charles Snyder....1
E. Viens....1
J. J. Waddell....1
Carl B. Warren....1
Geo. W. Whitesides....1
Benjamin Wilk....1
Chas. P. Williams....1
R. J. Willson....1
M. O. Withey....1
E. B. Wood....1
T. Van Dyke Woodford....1

Each of 45 other ACI Members divides credit 50-50 with another Member.

Walter V. Allen
D. M. Asarpota
P. G. Bowie
J. M. Breen
Geo C. Britton
A. D. Ciresi
Herbert K. Cook
Theodore Crane
R. W. Crum
R. A. Crysler
G. E. Davis
R. E. Davis
Harmer Davis
R. F. Dierking
A. W. Dudley
John R. Dwyer
H. H. Edwards
W. J. Emmons
H. J. Gilkey
Homer M. Hadley
P. J. Halloran
Walter N. Handy
P. W. Helsley
M. Hirschthal
L. I. Johnstone
John V. Konlhaas
B. Leon
G. L. Lindsay
N. M. Loney
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Maurice C. Miller
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Charles O'Rourke
F. W. Paulson
Warren Raeder
H. H. Scofield
G. M. Serber
Roy T. Sessums
R. H. H. Stanger
A. L. Strong
Sanford E. Thompson
Calvin T. Watts
Kurt F. Wendt
E. P. H. Willett
R. W. Winters
J. C. Witt

Illinois Men Win ASCE Awards

For the first time in the history of the American Society of Civil Engineers its two annual awards have been made to the same institution—the University of Illinois. Dr. Ralph B. Peck and Dr. Nathan M. Newmark, both members of the University's civil engineering staff, will be the recipients.

One other university possesses two medals, won in different years. Illinois previously had received three medals in different years.

The honors will be formally bestowed in the ASCE annual meeting, in New York this month.

Dr. Ralph B. Peck, U. of I. research assistant professor of soil mechanics, is to receive the Norman medal, highest award of the ASCE, for his paper on "Earth Pressure Measurements in Open Cuts of the Chicago Subway."

Dr. Nathan M. Newmark, research professor in civil engineering, an ACI Member, now on leave of absence from the university for service with the army in the Pacific, is to receive the Croes medal, which since 1912 has been awarded by the ASCE to the paper "next in order of merit to the paper to which the Norman medal is awarded." Dr. Newmark's paper is on "Numerical Procedure for Computing Deflections, Moments and Buckling Loads."

A.C.I. Announces its 42nd Annual Convention: New York, Feb. 18-21, 1946

WPB Removes Cement Type Restrictions

The War Production Board announced that restrictions on portland cement, which formerly limited manufacture to three specified types, were removed November 25 through revocation of Order L-179.

When the order was originally issued in August, 1942, requirements for portland cement for military and other essential construction work were unusually high and were expected to go higher, WPB said. Total 1942 consumption was 185,000,000 barrels, the highest consumption figure on record. The order was designed to increase production of the three most commonly used types by prohibiting manufacture of modifications of these types. Other provisions of the original order, subsequently removed, had prohibited the earmarking of storage bins for individual customers. This restriction was intended to promote full utilization of all available storage space.

Total actual capacity of the cement industry is approximately 215,000,000 barrels per year, WPB Building Materials Division officials stated, and that Consumption in 1944 will amount to an estimated 40 per cent of this capacity (88,000,000 barrels); 1945 consumption, if present construction restrictions remain unchanged, is estimated at from 50 to 60 per cent of capacity.

(*Adv.*)

WANTED—Superintendent for concrete products plant specializing in Architectural Concrete Slabs, Cast Stone, Pre-Cast joists and Specialty Products, must be versed in blue print reading, concrete mix designs, production schedules and capable of handling men. Will offer interest in business to right man. Annual volume well over quarter million dollars. Please state age, previous experience and salary desired. Address Box AA, c/o American Concrete Institute, Detroit 2, Mich.

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in 40 annual volumes of ACI Proceedings (since 1929 issued periodically in the Journal of the American Concrete Institute) and in many separate publications.

Recent ACI Standards

Building Regulations for Reinforced Concrete (ACI 318-41)

63 pages in covers: 50 cents per copy. (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

28 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

4 pages: 25 cents per copy

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

24 pages in covers: 50 cents per copy (40 cents to ACI Members)

Specification for Cast Stone (ACI 704-44)

4 pages: 25 cents per copy

Specifications for Concrete Pavements and Bases (ACI 617-44)

30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Proposed Standards

The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces

Reported by Committee 616 as information and for discussion only. 20 pages, 25 cents per copy (Reprint from ACI JOURNAL, June 1942)

Proposed Recommended Stresses for Unreinforced Concrete

Reported by Committee 322 as information and for discussion only. 4 pages, 25 cents per copy. (Reprint from ACI JOURNAL Nov. 1942)

Proposed Recommended Practice for the Construction of Concrete Farm Silos

Reported by Committee 714 as information and for discussion only. 16 pages. 25 cents per copy. (Reprint from ACI JOURNAL, Jan. 1944)

Proposed Minimum Standard Requirements for Precast Concrete Floor Units

Reported by Committee 711 as information and for discussion only. 16 pages, 25 cents per copy. (Reprint from ACI JOURNAL, Feb. 1944)

For further information on ACI Membership and Publications address

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Sources of Equipment, Materials and Services

A reference list of advertisers who participated in the Third Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1944 issue and (when it is completed) in V. 40, ACI Proceedings. **Watch for the 4th Annual Technical Progress Section in the February 1945 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 44th St., Alpena, Mich. 349
—Concrete products plant equipment

Stearns Manufacturing Co., Inc., Adrian, Mich. 339
—Vibration and tamp type block machines, mixers and skip loaders.

Construction Equipment

Baily Vibrator Co., 1539 Wood St., Philadelphia 2, Pa. 352
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 340-1
—Truck mixer loading and bulk cement plants, road building equipment, buckets

Butler Bin Co., Waukesha, Wis. 379
—Central mix, ready-mix and bulk cement plants, cement handling equipment

Chain Belt Co., Milwaukee, Wis. 376-7
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 366-7
—Concrete vibrators

Fuller Co., Catasauqua, Pa. 343
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio. 388-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio. 344-5
—Concrete paving equipment

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 368-9
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 347
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 365
—Concrete vibrators

Master Vibrator Co., 200 Davis Ave., Dayton, Ohio. 394
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 378
—Paving mixers

Smith Co., The, T. L. 2897 No. 32nd St., Wilwaukee 10, Wis. 374-5
—Mixer and truck mixer efficiency

Viber Co., 726 So. Flower St., Burbank, Calif. 355
—Concrete vibrators

Whiteman Manufacturing Co., 3249 Casitas Ave., Los Angeles 26 Calif. 392-3
—Floor construction and finishing equipment

Contractors, Engineers and Special Services

page

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 370-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 381-4
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 354
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago, Ill. 342
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 353
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia, 4, Pa 357
—Suction control of water in the concrete

Westberg Co., The, 611 No. Alvardo St., Los Angeles 26, Calif. 362-3
—Pneumatically applied concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 372
—Calcium chloride

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Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn, N. Y. 348
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—Waterproofing and densifier

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—Concrete curing compound

United States Rubber Co., 1230 Sixth Ave., Rockefeller Center, New York 20, N. Y. 373
—Form lining

Special ACI Publications

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members. $1 00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

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A Working Hypothesis for Further Studies of Frost Resistance of Concrete*

By T. C. POWERS†
Member American Concrete Institute

SYNOPSIS

Basic information is given on the freezing of water in concrete. From this information and other published material an explanation of the mechanism of the action of frost on concrete is developed. The explanation takes into account such factors as the degree of saturation of the concrete, the permeability and strength of the concrete, hydraulic pressures generated during freezing, and air-filled cavities. It is suggested that the hypothesis be made the basis of further laboratory studies of the action of frost in concrete.

INTRODUCTION

Laboratory testing of concrete to predict its ability to resist frost action has been carried on for many years. Nevertheless, no generally satisfactory testing technique has been developed; different laboratories testing the same kind of concrete often obtain different results and even in a given laboratory seemingly contradictory results are not uncommon. Thus, we still have a problem of developing a suitable testing technique and basis for interpretation of the results.

The writer has no ready-made solution of this problem to offer, but from the writings and experiments of others and from fundamental studies carried on in this laboratory it is possible to set up a working hypothesis which, together with other hypotheses, may eventually lead to the desired solution.‡ To the writer a working hypothesis such as this is not something to be accepted because it appears reasonable or rejected because it conflicts with previously acquired concepts and therefore appears unreasonable. Rather, it should be a logical development of certain major premises. The development should be such as to point out various implications of the premises and thereby to suggest experi-

*Received by the Institute Dec. 30, 1944.
†Manager of Basic Research, Portland Cement Assn., Chicago
‡Many of the ideas presented here are elaborations of those given by W. C. Hansen (Office memorandum, P. C. A. Research Laboratory).

mental procedures for testing the premises. The hypothesis presented in this paper rests mainly on the premise that the destruction of concrete by freezing is caused by hydraulic pressure generated by the expansion accompanying freezing of water rather than by direct crystal pressure developed through growth of bodies of ice crystals. If this premise is a fact, then various other things brought out in the discussion should be true. But, obviously, if the premise is false, the deductions are likewise false. It is hoped that attempts will be made to test the premises experimentally. Such attempts should lead to new approaches in the study of durability; if they do, the purpose of this paper will have been accomplished.

The hypothesis rests partly on certain as yet unpublished data on the structure of hardened portland cement paste and on the properties and behavior of the evaporable water that the paste may contain. It is necessary, therefore, to set down this information in order that the hypothesis may be correctly understood. No attempt will be made here to justify or qualify the various statements made about the structure of the paste; that must be done in other papers. Consequently, the discussion will unavoidably appear to be somewhat dogmatic.

Structure of cement paste

The hydration products are visualized as a porous gel enveloping unhydrated clinker and some non-colloidal hydration products, principally calcium hydroxide.* The porosity of the paste presents two aspects: First, there is the porosity of the gel-substance itself; second, there is the residual space unfilled by gel. The average size of the pores in the gel-substance is indicated as being about 8 times the diameter of a water molecule. When this porous gel-substance, together with the other solids, completely fills the available space, the only pores remaining in the paste (except entrained bubbles) are the extremely minute pores of the gel itself. When, however, the gel does not fill the original space completely, as is always the case in practical concrete, the paste is permeated by a system of submicroscopic, interconnected capillary spaces that are thought of as residues of the originally water-filled space. The exact manner in which the gel is laid down is not yet clear, but at present it seems most probable that the gel deposits mainly in the regions nearest the original cement particles and that therefore any residual space will be in regions that were at greatest distances from the original cement grains. The pattern of the capillary system is therefore determined by the original paste structure.

In the course of hydration the capillaries, at first full of water, become partially emptied as a result of contractions in volume that accompany

*L. T. Brownmiller, "The Microscopic Structure of Hydrated Portland Cement," ACI JOURNAL, Jan. 1943; *Proceedings* V. 39, p. 193.

hydration, studies of which have been reported from this laboratory and others.* They do not readily refill by absorption of water from the outside even though the specimen may be stored under water.

When small particles of cement paste are dried, the particles undergo a permanent, irreversible shrinkage. The shrinkage is such that in a paste having a water-cement ratio of about 0.5 by weight the hardened paste can reabsorb only 95 percent of the volume of water it was originally capable of holding. On the other hand, when a specimen of *concrete* containing a similar paste is dried, it is capable of reabsorbing more water than it lost initially. (This is probably due to the restraints against shrinkage offered by the aggregate particles.) Nevertheless, a specimen does not become fully saturated by absorption alone, probably because the extremely dense material desiccated during hydration is difficult to penetrate, and because of entrapped air.

The partial emptying of the capillaries during hydration is believed to be an important factor contributing to whatever degree of resistance to frost a concrete may show. This conclusion is based on the results of experiments which showed that when the air is removed from a dried specimen before introducing water, thus permitting the water to saturate the paste, a specimen normally able to withstand 100 or more cycles of freezing and thawing will disintegrate completely in less than 5 cycles. *Therefore, it may be concluded that when a specimen known to contain freezable water is able to withstand many cycles of freezing and thawing, its water content was initially below some critical value, that may be called the critical degree of saturation.* This, of course, is the same idea that has been set forth by Kreüger† and others.

As has been found for other porous building materials, a specimen of concrete will fail completely on freezing when its degree of saturation exceeds some critical value that is probably somewhere near 90 percent of saturation; but, as will be shown presently, concrete is of such nature that it can be damaged by repeated freezings before it reaches the critical degree of saturation.

Freezing temperatures

The highest temperature at which ice can exist in concrete depends on the degree of saturation of the concrete, or more fundamentally, on the forces acting on the freezable water. The magnitude of the forces is indicated by the relative vapor pressure of the evaporable water in the concrete. When concrete is saturated, or virtually so, the melting point is only slightly below 32 F, the amount of depression depending

*F. M. Lea and C. H. Desch, *The Chemistry of Cement and Concrete* (New York: Longmans, Green & Co., 1935), p. 175; W. C. L. Hemeon, "Setting of Portland Cement," *Ind. Eng. Chem. 27*, 694 (1935); T. C. Powers, "Absorption of Water by Portland Cement Paste during the Hardening Process," *Ind. Eng. Chem. 27*, 790 (1935).

†H. Kreüger, Transactions of the Royal Swedish Institute for Scientific-Industrial Research, No. 24, 1923, Stockholm.

on the concentration of soluble salts in the freezable water. When a saturated, hardened paste is cooled sufficiently, water held in the hardened paste will freeze, but unlike pure water in bulk, not all of it will freeze at the same temperature. This, of course, would be the effect expected of dissolved salts, but the effect is very much greater than can be accounted for by dissolved salts and is in fact a direct effect of the colloidal nature of the hydration products. This effect is such that the amount of ice formed in a given saturated specimen is an inverse function of the temperature, the lower the temperature the greater the amount of ice formed.

On raising the temperature, the ice disappears progressively and, in a saturated specimen, will be completely melted at a temperature slightly below 32 F.

Amount of freezable water

By a dilatometer method to be described in another paper the amounts of freezable water in various saturated pastes have been measured in this laboratory. The results can be expressed in the following general form:

$$w_f = w_t - kw_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where w_f = weight of freezable water at a given temperature
w_t = weight of total water in sample, including non-evaporable water
w_n = non-evaporable water
k = a constant which varies slightly with the type of cement and with the freezing temperature.

The non-evaporable water, w_n, is defined as that which exhibits vapor pressure less than about 4×10^{-4} mm of mercury at 23 C. This is the vapor pressure of the system $Mg(ClO_4)_2.2H_2O + Mg(ClO_4)_2.3H_2O$ at 23 C. w_n is about 10 percent more than the amount of "fixed water" held by a sample oven-dried at 105 C under average atmospheric conditions.

From Equation 1 it follows that when $w_f = 0$, $w_t = kw_n$. We may designate this particular value of w_t as w_t'. For a given cement and freezing temperature w_t' is thus seen to be proportional to the extent of hydration of the cement.

The proportionality factor k was evaluated from tests made on five different cements. The data were obtained from samples taken from a group of specimens comprising three different mixes (w/c = 0.3 to 0.6) and two curing periods (28 and 90 days) for each cement. The average value of k was obtained by a short extrapolation of a straight-line plot and was found to be 1.75. This value shows that for a saturated paste to be free from water freezable at − 15 C, which is virtually all the freezable water, the total water content must not be more than about 1.75 times

the non-evaporable water content. This means also that the *un*freezable part of the evaporable water is a quantity equal to about ¾ of the non-evaporable water content.

Since the total water content of a saturated paste is limited by the original water-cement ratio, the above figures indicate that if the original water-cement ratio is properly limited the paste will be free from freezable water after hydration. The limit will be higher the greater the extent of hydration. The original water ratio is not indicated directly by these figures, however, since the total water content includes the gain that occurs during the curing period and is therefore larger than the original water content. For estimating the amount of freezable water in terms of the original water content, w_o, the following empirical relationship was found to hold:

$$w_f = m(w_o - K\,w_n) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where m is an empirical constant. Its average value was found to be 1.16. K has the same significance as k in Equation 1 but is numerically smaller. When $w_f = 0$

$$w_o' = K\,w_n$$

where w_o' is the original water content of a paste having no freezable water after hydration. The average value of K was found to be 1.3.

The values of w_o' can be determined from the extent of hydration as measured by the non-evaporable water content. The value so determined for cements of average composition was found to be about 0.30 g per g of cement for specimens cured 28 days, and about 0.32 g per g of cement for those cured 90 days, the average non-evaporable water contents being 0.230 and 0.246 g per g of cement, respectively.

These values indicate the maximum original water-cement ratios for pastes having no freezable water after hydration. Since all practical concrete is made with higher water-cement ratios than these, it contains, or is capable of containing, freezable water. The quantity of freezable water can be estimated from Equation 2. For purposes of estimation, this may be written

$$w_f = 1.16\,(w_o - 1.3\,w_n) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For example, if $w_o/c = 0.5$,

then $\qquad w_f/c = 1.16\,(0.5 - 1.3\,w_n/c)$.

When $w_n/c = 0.2$, $w_f/c = 0.28$ g per g of cement.

The data given above all pertain to samples cured at normal temperatures. If a high curing temperature is used, the relationships are much different from those just described.

Situation of ice in concrete

The spaces in undamaged concrete that may absorb and hold water are believed to be as follows:

(1) Spaces within the gel substance itself;
(2) The capillary system within the mass of gel substance;
(3) The capillary system in absorbent aggregate particles.

Analysis of adsorption isotherms and studies of the density of the evaporable water (to be reported in other papers) both indicate that in a saturated paste, a quantity of evaporable water 0.75 to 1.0 times the quantity of non-evaporable water is "gel water" as distinct from "capillary water."* As shown above, independent measurements of freezable water showed that the *un*freezable water is an amount equal to about 0.75 times the amount of non-evaporable water. This may be considered as experimental evidence that the ice formed in the paste occurs only in the capillary system within the gel substance. The capillaries are thought of as residues of the original water-filled space not filled by gel and hence are probably pocket-like but apparently interconnected.

Besides this experimental evidence there are some indirect indications that ice is not likely to form within the gel substance. The average specific volume of the water in the gel is about 0.85. This indicates that all or most of it is within the range of the surface forces of the solid phase and that therefore the spaces in which the water is held are exceedingly small. In fact, there is reason to believe that the relationship between the gel water and the gel substance is not much different from solid solution. The probability of nuclei formation in water held in such a state is much lower than it is in the larger capillary spaces where, when the paste is saturated, the water is under no stress.

Ice probably does not form in the macro- or even microscopic cavities of an undamaged specimen. Such cavities, surrounded by absorbent paste, can contain no water until some force such as that arising from expansion on freezing displaces water from the surrounding paste into the cavity.

Besides forming in capillaries within the paste ice may form also in the capillary system of absorbent aggregate particles.

MECHANISM OF DISINTEGRATION

When concrete specimens are frozen and thawed in water, two distinct types of disintegration have been observed: (1) The specimens may show little change in weight and appearance but large losses in strength and resilience; or (2) they may show progressive loss by crumbling and spalling but relatively little loss in strength and resilience of the remaining material. These are extremes. Specimens falling in neither category will show both crumbling and loss of strength and resilience of

*Evaporable water is defined as that having a vapor pressure at 23 C. greater than about 4×10^{-4} mm of mercury.

the remaining material. These are results that an acceptable hypothesis must explain satisfactorily.

Generation of hydraulic pressure

Ordinary ice can exist under pressures up to about 29,000 psi., but of course not at 32 F. At 32 F the pressure cannot exceed 1 atmosphere, but for each Fahrenheit degree drop in temperature below the normal melting point the pressure may be increased about 736 psi., up to the maximum mentioned, which is reached at about −4 F.* For example, at 13.5 F ice can exist under a pressure of about 10,000 psi.; or, to put it another way, a pressure of about 10,000 psi. would be required to prevent the formation of ice at 13.5 F if conditions are otherwise such that the ice could exist at 32 F under a pressure of 1 atmosphere.

The above remarks give the magnitude of pressures that would be exerted on a piston closing a water-filled cylinder if the piston were held so as to prevent the expansion that accompanies freezing. It is thus indicative of the magnitude of the pressure that might develop in a completely saturated, sealed specimen of concrete.

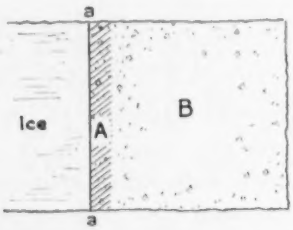
Experiments show that fully saturated concrete (saturated with the aid of a vacuum pump) is not able to withstand the pressures that are developed at the low temperatures used in freezing and thawing tests. This shows that concrete is not sufficiently extensible to undergo the strains that would be necessary to relieve the pressure. Hence, we may conclude that since *apparently saturated* concrete does not ordinarily fail completely on first freezing it is not fully saturated, even after prolonged soaking. That is, ordinarily there is enough residual space in the concrete to accommodate the expansion that accompanies freezing. Nevertheless, under certain conditions some specimens show loss of resilience and abnormal expansion on repeated freezing and thawing even though they are not fully saturated; that is, they are damaged by freezing even when the degree of saturation of the specimen as a whole is below the critical value that would cause bursting and complete disintegration. For a working hypothesis to be satisfactory it must account for this fact. The following discussion is an attempt to meet this requirement.

Consider a surface of a specimen that has been in contact with water for some time prior to the beginning of the freezing cycle. The water content of the concrete near the surface is probably at or near total saturation, and is higher, though perhaps only slightly, than the average water content of the specimen. If this surface is so situated that heat from the concrete flows through it to the surrounding water, the sequence of events should be as follows: First, the water against the surface will

*N. E. Dorsey, *Properties of Ordinary Water Substances* (Reinhold Publishing Corp., 1940), p. 467.

freeze, thus sealing the surface of the concrete; second, the water in the capillary spaces of the concrete nearest the surface will freeze and as the change of state takes place the still unfrozen water in the saturated region will be displaced towards the less saturated interior. This may be visualized with the help of Fig. 1. The drawing represents a cross-section of a part of the specimen, the surface in question being normal to the plane of the page at aa. A is the saturated region near the surface; B is the region of lower water content.

Fig. 1



Ice will form on the outside of the surface before any will form in the specimen because of the depression of the freezing point mentioned above and perhaps because of the tendency for the water in the concrete to supercool. Freezing will occur in A before it does in B, because of the direction of the temperature gradient, and because A is at a higher water content than B and hence because the water in A can freeze at a higher temperature than the water in B.

When ice begins to form in region A, the unfrozen water will be displaced toward the less saturated region B. If the water were free to move without resistance, no hydraulic pressure whatever would develop. However, since the water is required to move through a fine-textured, porous substance, the force causing the movement will give rise to a corresponding frictional (viscous) resistance and gradients of hydraulic pressure will be present during the movement of the water according to the laws of hydraulic flow.

If this reaction against the force displacing the water inward is sufficiently high, then it can be regarded as being capable of damaging the specimen. The possible magnitude of the hydraulic pressure can be estimated on the basis of published data on the permeability of concrete. For concrete having a water-cement ratio of 0.6 by weight, cured 28 days, the coefficient of permeability is about 40×10^{-12}.* Hence, by Darcy's law,

$$Q = 40 \times 10^{-12} \times \frac{H}{L}$$

*A. Ruettgers, E. N. Vidal and S. P. Wing, ACI JOURNAL, Mar.-Apr., 1935; *Proceedings* V. 31, p. 382. NOTE: The unit of permeability used here is cu. ft. of water per sq. ft. of concrete per second per foot of head per foot of thickness.

where Q = rate of flow through the concrete, ft. per second
H = pressure-drop through the layer, ft. of water
L = thickness of layer, ft.

If we assume a given rate of water movement (corresponding to a given rate of freezing), then we can compute the required difference in pressure over a given thickness of the region through which the water is being forced. However, for the present purpose it is advantageous to estimate the rate of movement required to develop a reacting pressure sufficient to damage a specimen of concrete. If we assume that the concrete has a tensile strength of 500 psi. and that the mobile water contacts 10 percent of the cross-section, then it would follow that a hydraulic pressure of 5000 psi. would be required to produce an average stress equal to the strength of the concrete. However, the specimen would probably be damaged at a lower hydraulic pressure since stresses adjacent to the source of pressure would be well above the average. It seems safe to assume that a pressure of 2500 psi. would be destructive. (Such a pressure could be developed at any temperature below about 28.5 F.)

With this assumption regarding the magnitude of a damaging pressure we can compute the velocity with which the water must be forced through a given thickness to give rise to reacting hydraulic pressure of destructive magnitude. If the layer through which the water moves is 0.001 ft. (0.012 in.), then to develop a reacting pressure of 2500 psi. (5800 ft.-head), the rate of flow would be

$$Q = \frac{40 \times 10^{-12} \times 5800}{10^{-3}}$$

= 232 millionths of a ft. per sec.
= 10 inches per hour.

A rate of flow of 10 inches per hour would correspond to a rate of ice formation of about 100 inches per hour. Such a high rate of freezing is improbable, even under laboratory conditions; hence, if damage results from freezing through the resistance to movement of water, the thickness of the layer through which the water is forced to move must be considerably in excess of that assumed in the above computation, 0.012 inches. Since the velocity necessary to develop a given reacting pressure is inversely proportional to the thickness of the region through which the water is being forced, the required rate for different thicknesses can be readily seen from the above computation. For example, if the thickness is 10 times that assumed, 0.12 instead of 0.012, the rate of flow would be 1 inch per hour and the rate of freezing about 10 inches per hour; or if the thickness were 0.24 inches, the rate of water movement would be ½ inch per hour and the rate of freezing, 5 inches per hour, and so on.

Depths of saturation in excess of ¼ inch and rates of freezing of the order last mentioned are well within the range to be expected in many laboratory freezing tests. Hence, to consider the reacting hydraulic pressure accompanying water movement as a possible source of disruptive pressure is in line with experimental data on the tensile strength and degree of permeability of concrete. It indicates a possible mechanism whereby a specimen of concrete can be damaged by freezing even though it may not be saturated to the critical degree.

In connection with the frost heaving of soils, Stephen Taber* has described another mechanism of pressure development in porous bodies that are not fully saturated. In soils that are not too highly impermeable or that are not carrying too great a superimposed load or that are not cooled too rapidly, pressure is developed by the formation of segregated bodies of ice crystals which grow in a direction parallel to the heat flow by receiving water from the unfrozen interior. This theory will be discussed more fully in a later section of this paper. At this point it will suffice to say that the writer believes that the same reasoning and experiments which demonstrated the soundness of Taber's theory with respect to the behavior of certain soils leads to the conclusion that the same phenomenon is not likely to occur in concrete, at least under the conditions of laboratory freezing and thawing tests. Consequently, the hypothesis here developed is based on the assumption that hydraulic pressure is the primary disruptive force. In Taber's language, the assumption is that concrete behaves as a closed system whereas some soils behave as open systems; and the thought is here added that in a porous body which acts as a closed system disruptive back-pressure may be developed by motion of the water even when there is enough room within the body to accommodate the total expansion. However, the phenomenon described by Taber is probably not entirely absent, as will be brought out later.

Crumbling and spalling

On repeated freezing and thawing in water, region A should increase in thickness according to the amount of water absorbed by the specimen. As the thickness of region A increases, the resistance to displacement of water out of that region toward the region of lower water content will increase, and when the saturated region becomes sufficiently thick the hydraulic pressure will become greater than the strength of the material and cause disintegration or spalling of some part of region A. The thickness of region A at the time when disintegration in region A begins is as *the critical depth of saturation* mentioned before.

The concept of critical depth of saturation as given above is a somewhat oversimplified picture, for in reality the saturated region is not

J. Geol. 37, 428 (1929); *38*, 289 (1930); *Public Roads 11*, 113 (1930).

sharply separated from the other; instead, the two regions are joined by a transition zone where there is a continuous moisture gradient. Moreover, the saturated region is seldom if ever completely saturated; even after prolonged soaking it contains spaces into which water can be forced by high pressure. However, the simplified version will serve the purpose of this discussion.

One of the implications of this concept is that if a specimen were uniform in structure (homogeneous, in a restricted sense) and not wholly saturated at the start, no crumbling or spalling would result from freezing until a certain amount of water had been absorbed, such as to saturate the surface region to the critical depth.

The critical depth of saturation should be different for different grades of concrete because the magnitude of the hydraulic pressure and the average stress produced will depend on the following factors:

(1) The hydraulic pressure will depend on:
   (a) The permeability of the material through which water must flow to escape from the saturated region.
   (b) The rate of freezing.
   (c) The amount of water in region A in excess of the critical degree of saturation.

(2) At a given hydraulic pressure the average stress in the concrete will depend on the proportion of solids in a unit cross-section.

The degree of permeability (item (1a) above) should have a marked effect on the intensity of hydraulic pressure generated in the saturated region during freezing. The following illustration will show how this comes about:

The amount of freezable water in saturated paste can be estimated from the empirical relationship given before:

$$w_f/c = 1.16\ (w_o/c - 1.3\ w_n/c)$$

For an average cement cured 28 days, w_n may be assumed to be 0.22. Hence,

$$w_f/c = 1.16\ (w_o/c - 0.29)$$

With the aid of this relationship the following table was prepared:

Properties of the Concrete (Estimated or Assumed)	Rich Mix	Lean Mix
Cement content, lb. per cu. yd.	610	390
w/c by weight	0.45	0.70
Freezable water content at saturation:		
lb. per cu. yd.	115	186
cu. ft. per cu. yd.	1.85	2.98
cu. ft. per cu. ft.	0.068	0.110
Coefficient of permeability*	2×10^{-12}	150×10^{-12}
Tensile strength, psi	600	300

*Ruettgers, Vidal, and Wing; *ibid.*

If water is forced through a thickness of 0.001 ft. at the rate of 1 inch per hour (= 23.2×10^{-6} ft. per sec.), the respective pressures developed as computed from Darcy's law are:

Rich concrete	5020 psi
Lean concrete	67 psi

These figures are, of course, significant only on a relative basis, since the rate and thickness are arbitrarily chosen figures. However, since the rate of freezing is probably not appreciably influenced by the quality of the concrete, and since a given distance of flow could be found in either quality, the pressures indicated should be in substantially correct ratio.

To illustrate how the permeability and strength determine the critical depth of saturation, the following computation is given, based on an assumed rate of water-movement of 0.1 in. per hour.

The average stress in each concrete will be
(unit-hydraulic-pressure) × (fractional area contacted by the mobile water)
or $502 \times 0.068 = 34.0$ psi for the rich concrete
and $6.7 \times 0.110 = 0.7$ psi for the lean concrete,
this computation being based on a depth of saturation of 0.001 ft. Since, at a given rate, pressure is inversely proportional to depth of saturation, the depth of saturation at which the average stress equals the tensile strength is

$(600 \div 34.0) \times 0.001 = 0.0176$ ft. $= 0.21$ in.

for the rich concrete, and

$(300 \div 0.7) \times 0.001 = 0.43$ ft. $= 5.2$ in.

for the lean concrete.

It must be emphasized that the absolute values given may be far from the actual facts; they serve only to illustrate the principle on which the hypothesis is based. For example, if the actual rate of movement is 1/10 that assumed, the thickness required for a given average stress is 10 times the value given. It must be borne in mind also that the computations rest on the assumption that the region where ice is forming is fully saturated. This condition is probably never fulfilled; hence, the computations represent a limiting case.

These figures might seem to indicate that a rich concrete should show more crumbling or spalling than a lean one, since in this case the critical depth of saturation of the rich mix is only one-twenty-fifth that of the lean; it looks as if the rich mix might show surface disintegration earlier than the lean one. However, to reach such a conclusion would be to ignore common experience and to overlook the great difference between the rates of absorption of such concretes; the lean concrete absorbs water much more rapidly. Moreover, rich concretes do not become fully saturated, even on prolonged soaking, unless capillary action is augmented by the forces produced on freezing. Note also that even if it took similar lengths of time for the two grades of concrete to become saturated

to their respective critical depths, the loss per cycle per unit area from the leaner concrete would be much greater than that from the richer.

It is important to note in this connection that under natural conditions of exposure, concrete is usually subjected to alternate wetting and drying as well as freezing and thawing. If the periods of drying are long relative to the period of wetting, dense, impermeable concrete manages to remain at a low percentage of saturation as compared with that of low-grade concrete which absorbs water quickly. Moreover, impermeability is usually the result of using a high cement content and hence such concrete reaches a higher degree of desiccation through the contractions that accompany hydration.* These advantages, particularly the advantage of a low rate of absorption, are not always brought out in laboratory tests where small specimens are used.

Item (1c) above concerning the degree of saturation requires special comment. Theoretically, if the water content of region A is about 90 percent of saturation or less, then all the expansion can be accommodated without any of the water escaping from A. If the water content exceeds 90 percent of saturation, then the excess must escape or region A will disintegrate; the greater the excess the greater the amount of water that must escape in a given length of time; hence, the greater the hydraulic pressure developed.

If in a given specimen the critical depth of saturation happens to be equal to or greater than half the thickness of the specimen, then disintegration should not begin until the entire specimen reaches the critical state of saturation. Such a specimen should show little or no disintegration during the cycles required to bring it to the critical state, but when that state is reached the specimen should disintegrate almost completely in a few final cycles. This apparently accounts for the observed behavior, mentioned earlier, of those specimens that show little change in weight or appearance, but, in a final few cycles, rapidly lose all strength and resilience. Such specimens are the relatively coarse-textured ones of high permeability.

Damage without disintegration

The critical depth of saturation discussed above pertains to the depth to which the surface region must be saturated before surface disintegration takes place. Extension of the same principles cited before leads to the conclusion that concrete may also be damaged even in regions where expansion on freezing could be accommodated by available space within the region.

*For a discussion of the importance of this phenomenon see M. A. Swayze, "Early Concrete Volume Changes and Their Control," ACI JOURNAL, Apr. 1942; *Proceedings* V. 38, p. 425.

The disruption of porous, water-laden aggregate particles is sometimes responsible for this; water in a particle is trapped by ice or by relatively impermeable paste surrounding the particle, and the hydraulic pressure generated by freezing disrupts the particles and expands the specimen. However, a similar result in the absence of such aggregate particles is conceivable. Even if the paste is not saturated to the extent where pressure causes disruption, the pressure may still be high enough to cause some dilation. Such an effect in a low-w/c concrete might develop at a lower degree of saturation than in a high-w/c concrete. This is suggested by the figures given above on relative intensities of hydraulic pressure as controlled by permeability. In the example given, it can be seen that a condition that would develop negligible pressure in a lean mix might develop pressures high enough to cause dilation of a richer mix.

Some experimental evidence supporting this is given in Fig. 2. This represents 3x3x15-in. concrete prisms (0-¾ in. aggregate) that had been cured in the moist room 7 days, stored in air at 50 percent relative humidity for 6 weeks, and then soaked in water 4 days just before the beginning of the freezing and thawing test. Note that the richest specimens showed relatively little loss in weight but relatively high expansion and drop in Young's modulus. For example, after 100 freezings and thawings, the concrete prisms containing 4 sacks of cement per cubic yard showed 0.1 percent increase in length and those containing 7 sacks showed 0.3 percent increase. The apparent decreases in Young's modulus were 28 and 65 percent, respectively. However, the surface disintegrations compare quite differently; the weight loss of the leaner concrete was about 8 percent and that of the richer, 2 percent. The relative weight losses

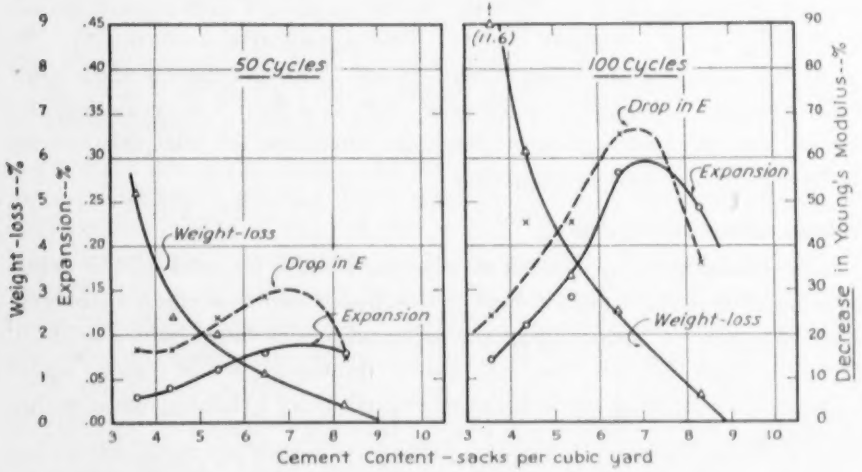


Fig. 2—Results of freezing and thawing tests of concrete having 3 to 4 inch slump

seem to reflect the relative permeabilities and hence rates of absorption, as would be expected, but the relative expansions and losses in resilience seem to require the explanation given above.

These particular specimens contained some unsound chert and thus a true explanation may have to include this factor also. However, a smaller group of tests using wholly sound aggregate gave results similar to these. Hence, it seems very doubtful that the small amount of disruptive aggregate can account for the result shown. These data therefore are regarded as showing the effects of dilation of the paste itself.

Influence of cavities

The discussion up to this point has carried the tacit assumption that the submicroscopic pores in the partially desiccated paste are the only spaces available for the movement of water during freezing. If this were the usual condition, concrete would show a very inferior resistance to weathering. Ordinary concrete contains many air-filled cavities that are not in direct communication with the surface. These cavities are entrained air bubbles, accessible pores in the aggregate particles, and thin fissures under the aggregate particles. The fissures, formed during the period of bleeding, are at first water-filled but may become partially or wholly emptied as hydration proceeds. Such fissures are more numerous and larger the leaner the concrete and the greater the slump of the fresh mix.

All empty cavities of the kind mentioned, especially air bubbles, are difficult to fill with water. They cannot be filled by capillary action (by soaking) because a liquid cannot spontaneously flow from a small capillary to fill a larger one. However, water can be forced into a cavity by applying external pressure or by pumping the air out of the previously dried specimen before admitting water. Under normal conditions a pressure exceeding one atmosphere is required to fill a cavity, because as the cavity fills its contained air becomes compressed. The pressures generated during freezing are probably more than sufficient to force water into such spaces.

It might at first seem that the amount of space required to protect concrete would depend on the amount of expansion to be accommodated. However, this is not the case, as will now be explained.

As shown in the examples previously given under "Crumbling and Spalling", the freezable water content of saturated concrete would ordinarily lie between 0.07 and 0.10 cu. ft. per cu. ft. of concrete, and the expansion of the water on freezing would be roughly 0.007 to 0.010 cu. ft., or 0.7 to 1.0 per cent of the volume of the concrete. Therefore, if the concrete contains 0.7 to 1.0 percent air-space, the expansion can,

theoretically, be accommodated. If the space is air-filled at a pressure of one atmosphere, and if we assume that the concrete can withstand 170 atmospheres (2500 psi.) of hydraulic pressure, then we need to provide space equal to the volume expansion plus 1/170 of that volume for compressed air which is negligible. That is, we need allow space equal to only slightly more than 0.7 to 1.0 percent of the volume of the concrete to accommodate the expansion. Since concrete ordinarily contains at least this much air-filled space, it should be able to withstand freezing and thawing almost indefinitely. Such a conclusion is obviously contrary to fact.

The figures just considered show that if the destructive action of freezing is due to hydraulic pressure, the resistance to *movement* of water must be the primary source of pressure, for practically all concrete contains enough air-filled space to accommodate the water-to-ice expansion. Hence, the distance between air spaces rather than the total volume of air space would be the factor determining the degree of protection, provided the total space is at least 1 percent or thereabouts.

As will be seen further on in this discussion, the destructive expansion could be accounted for by a modification of Taber's hypothesis, that is, by assuming that the damage is caused, not by hydraulic pressure, but by the growth of submicroscopic bodies of ice analogous to ice lenses. This would account nicely for the fact that concrete containing enough space to accommodate the ice-to-water expansion may nevertheless be damaged on freezing. However, it would fail to provide a very convincing explanation for the fact that introducing more air spaces prevents or greatly reduces damage by frost action. This is a reason for attempting to develop a hypothesis from a premise not involving direct crystal pressure.

To this phase of the hydraulic-pressure hypothesis there is the following important corollary:

Given a total air space greater than the possible amount of expansion, the protection of the concrete will be greater the smaller the average size of the individual air spaces.

This follows from the fact that for a given volume of air held within prescribed boundaries, the air being divided into a large number of separate units, the greater the number of units the closer together they must be.

The equally important converse of the corollary is this:

For a given degree of protection, the smaller the air-filled cavities the smaller the total volume of air required.

Experience shows that if concrete contains a large number of small, well distributed air cavities, as it does after it has been mixed in the

presence of a foam stabilizer, its rate of disintegration and rate of internal damage can be reduced to a small fraction of the usual rate. In terms of the matters discussed above, this is seen to be the result of providing ample room for expansion *and of limiting the thickness of the permeable material through which the water must move to find escape from ice pressure.* Other effects may be involved in this case, but they need not be considered in connection with this hypothesis.

This hypothesis provides an explanation for some of the results reported by Kennedy.* He found that neat cement prisms containing entrained air failed in a freezing and thawing test more rapidly than did a companion specimen containing virtually no air. The published photographs suggest that unlike air entrained in mortar or concrete, the air in these specimens collected in a relatively small number of large cavities separated by thick layers of paste. As can be seen from the above discussion, such cells would offer relatively little protection. In fact, once the wall of a large cavity is ruptured, water (or in this case $CaCl_2$ brine) can flow in and thus transform the cell from a source of protection into a source of disruptive pressure.

Relative permeabilities of paste and aggregate

If the aggregate particles in a specimen of concrete are less permeable than the paste, then they should increase the intensity of hydraulic pressure in the region where the paste is saturated, since, for a part of the freezable water, they block the most direct path to the unsaturated region. On this basis one would expect concrete containing impermeable aggregate to tend to fail along aggregate surfaces under the action of freezing. This has been pointed out previously by W. C. Hansen.†

If the concrete contains unsaturated aggregate particles that are more permeable than the hardened paste, those particles should moderate the hydraulic pressure somewhat as cavities do, *until the particles become saturated.* When the particles are saturated, water must escape into the surrounding paste during freezing or pressures will develop that are high enough to disrupt the aggregate particles and the surrounding material. The intensity of the pressure in saturated particles thus depends on the permeability of the paste that lies between the particles and the unsaturated region.

Influence of the initial degree of saturation

In an earlier part of the discussion it was stated that when concrete is partially dried, the initial freezing point or, more accurately, the final melting point is lower the greater the degree of dryness of the specimen. The final melting point is, in fact, a function of the relative vapor pressure

*Henry L. Kennedy, "The Function of Entrained Air in Concrete," ACI JOURNAL, June, 1943; *Proceedings* V. 39, p. 529.

†Office memorandum, Portland Cement Association, Research Laboratory.

of the water remaining in the specimen. Thus, the degree of saturation not only determines the severity of the effect of freezing, as discussed above, but also determines the maximum temperature at which ice can exist within the specimen.

A specimen that has been cured without gain or loss of water, that is, one that has been sealed after casting, would show a final melting point lower than the normal melting point of the "cement-solution" which it contains; the lower the water-cement ratio and the greater the extent of hydration the lower would be the final melting point in a specimen cured in this manner. Well hydrated, sealed specimens of low water-cement ratio probably contain no water that is freezable at temperatures normally experienced in this climate.

When specimens have access to an outside supply, they tend to absorb water during the course of hydration, as evidenced by their increase in weight. Specimens of high water-cement ratio are able to absorb nearly enough water to compensate for the contractions accompanying hydration; specimens of ordinary dimensions and low water-cement ratio are not able to do so except during the early stages. Thus, the resistance of rich specimens to *surface disintegration* is accounted for by their lesser degree of saturation.

Factors governing the amount of water absorbed during thawing

It seems fairly evident that the life of a specimen in a freezing and thawing test depends on its initial degree of saturation and to a marked degree on the rate at which it can absorb water. One of the factors involved in this is the mode and rate of thawing.

If the surface of a frozen specimen is covered with ice, the specimen cannot absorb water through that surface until the ice-coating melts. After the outside ice has melted, the rate of absorption should depend partly on the amount of pressure-deficiency, i.e., the negative pressure, within the specimen. This should depend on several factors, as will now be shown.

On freezing a specimen that is in contact with ice, hydraulic pressure is created that drives water inward or into cavities adjacent to the frozen regions, as already explained. When the temperature of a frozen specimen is raised, thawing inside the specimen begins immediately, no matter how low the temperature, and the last ice will melt at a temperature *below* the melting point of the ice covering the surface. The time available for this melting inside the specimen before the outside ice melts is believed to be a significant factor. If heat is received rapidly by the surface region, most of the outside ice may be melted before all the ice in the specimen disappears, owing to the steepness of the temperature gradient. However, if heat is received slowly, a layer of ice will remain

on the surface and thus maintain a constant temperature of 32 F for a considerable period.

Under these conditions of thawing, heat flows into the specimen through the outside layer of ice. Hence, melting within the specimen will first occur near the surface of the specimen and thus produce a layer of melt between the outside ice and the still-frozen interior. The *contraction* accompanying melting should create a negative pressure in this layer and thus as melting proceeds the water should flow toward this region. *As long as the outside remains sealed with ice. all such flow must be back from spaces into which the water was originally displaced.* Thus, in this circumstance the original distribution of the water tends to be restored during the period of thawing. If water from the outside becomes available before thawing on the interior is completed, *then water should be absorbed whether the specimen has become permanently dilated or not.* The rate of absorption will, of course, depend upon the difference in pressure and on the permeability of the material. Specimens of low water-cement ratio seem to absorb very little water during the thawing periods. However, existing data are unsatisfactory on this point because measurements of the weight of the specimen indicate only the net results of any losses due to disintegration and gains due to absorption of water.

The fact that the melting of the ice in the specimen may precede the melting of the outside ice probably is one important factor accounting for the resistance to disintegration or scaling of concrete containing an air-entraining agent. Because of the sequence of events as described, even the air-filled cavities just under the surface tend to be emptied before any water from the outside is available to them. If this were not true, it would be difficult to account for the resistance to scaling shown even by the troweled surface of concrete containing entrained air.

Freezing and thawing in $CaCl_2$ brine has been found to be more destructive than freezing and thawing in water. In terms of the hypothesis, a specimen frozen in brine should be able to absorb more liquid during the thawing period than one frozen in water; hence, the rate of disintegration in brine should be greater than in water. The greater absorption is due to the lower melting point of the brine. A 10 percent $CaCl_2$ brine, for example, will be fully melted at about 22 F. Since most of the ice within the specimen melts above this temperature, it follows from considerations already given that the portion of the thawing period during which liquid from the outside can be absorbed is relatively large for the specimens frozen in brine.

Experimental evidence that the amount of absorption per cycle in brine is greater than it is in water can be found in data published by

Hansen.* In Hansen's tests the specimens were given 100 cycles in water and then a 10 percent $CaCl_2$ brine was substituted for the water for the rest of the cycles. During the cycles in water many of the specimens showed very little change in weight after the first 5 cycles. As soon as the brine was substituted for the water, the amount of absorption per cycle for these specimens showed a marked increase. In this case, the specimens that withstood the first 100 cycles in water also stood up well in the subsequent cycles in brine, so that the increased rate of absorption observed was not clearly connected with an increased rate of disintegration. However, the result seems to account at least in part for the generally greater severity of the test when specimens are frozen in brine instead of water.

Effect of thickness of outside layer of ice

It is possible that the thickness of the ice layer formed on the outside of the specimen is one of the factors influencing the number of cycles of freezing and thawing required to produce a given degree of disintegration. This follows from the consideration given above: the thicker the layer of ice, the longer will be the period during which the temperature at the surface of the specimen remains at 32 F and therefore the more nearly will the pressure differences within the specimen produced on melting be equalized before outside water becomes available. To produce uniform results in a freezing and thawing test in which the specimens are surrounded with water, it therefore seems important to control carefully the amount of ice allowed to form around each specimen. If follows also that conditions of thawing should be carefully regulated so that all specimens receive heat at the same rate.

When the practice is followed of freezing the specimens in air and thawing them in water, a maximum of opportunity for absorption of water from an outside source is provided since there would be little if any ice on the surface to keep the specimen sealed while thawing. This would be a factor tending to hasten the process of surface disintegration. However, the lack of an ice-seal might reduce the effect of freezing, for pressures might be relieved by flow to the outside of the specimen as well as toward the interior.

Data published by Hughes† indicate that freezing in air and thawing in water is less destructive than freezing in water and thawing in water. This would indicate, according to the above paragraph, that the opportunity for water to escape to the outside while freezing in air protects the specimens more than the free access to water during thawing harms them.

*W. C. Hansen, "Influence of Sands, Cements, and Manipulation upon the Resistance of Concrete to Freezing and Thawing," ACI JOURNAL, Nov. 1942; *Proceedings* V. 39, p. 105 (1943).

†C. A. Hughes, "The Durability of Cement Mortars; the Cement and Method of Testing Major Variables," *Proc.* A.S.T.M., *33*, Part II, 511 (1933).

Nature of the negative pressure in concrete

When freezing occurs in a saturated region and water is forced from that region to another, the water content of the saturated region will be lowered about 10 percent, if no expansion of the solid occurs. If the region entered by the displaced water contained air, the air would be compressed. For example, with an air-entraining cement, any water forced into one of the many air-cells would compress the air in that cell. When thawing occurs, the contraction lowers the pressure in the thawed region below that in the cavity into which the water was forced, as explained before. Some of the pressure difference may thus be due to compressed air, but probably this is a small part of the total force acting; capillary force probably accounts for the greater part of the total. This may be shown by the following example based on experimental data.

Consider a saturated region containing one gram (=1 cc.) of liquid water. We will assume that $\frac{2}{3}$ g. of the water is freezable under the conditions of the test. Hence, the expansion on freezing will be $\frac{1}{10}$ of $\frac{2}{3}$, or 0.07 cc. When this amount of water is displaced from the saturated region, the water content of the region will thereby be reduced to 0.93 g. When melting occurs, the region will therefore appear to be "dried" to the extent indicated. From other data it is known that such a decrease in water content would be accompanied by a small lowering of the vapor pressure. A given lowering of the vapor pressure is indicative of the magnitude of capillary force tending to drive water toward the partially dried region. If, for example, the vapor pressure is 98 percent of the saturation pressure, the maximum capillary force is 28 atmospheres (400 psi.). Thus, if an air cell contained water having a relative vapor pressure of 100 percent and the surrounding region contained water having a relative vapor pressure of 98 percent, a counteracting pressure of 28 atmospheres would have to be exerted to prevent water in the cell from entering the region. It therefore seems possible that capillary effects determine the direction and rate of water movement during the thawing period.

Rate of freezing

As mentioned before, the intensity of the hydraulic pressure that is developed during the freezing of a not wholly saturated specimen depends on the rate of movement of the water, or in other words, on the rate of freezing. Hence, it would seem that the higher the rate of cooling the more destructive the effect of freezing.

The rate of freezing in any part of the specimen should depend upon conditions external to the specimen and will not be the same in all parts of the specimen. The rates in general will depend on the rate of cooling as controlled by the amount, kind, and temperature of the refrigerant,

the amount of water surrounding the specimen, etc. The rate of freezing at a point in the specimen should depend on the distance of that point from the surface, the greater the distance the lower the rate of freezing. Since the intensity of the generated pressure depends on the rate of freezing, it follows that the greater the distance of a point from a surface the less the effect of freezing. Consequently, if the rate of freezing is controlled by the rate of cooling (no supercooling), the size of the specimen could have a considerable influence on the test results; the larger the specimen the smaller the internal damage caused by overstraining.

The Taber-Collins hypothesis

As mentioned before, Taber advanced a hypothesis to account for the expansion of soils that differs materially from that set forth above for concrete. Very recently an application of Taber's theory to the frost resistance of concrete has been published by Collins.* Collins concluded from his observations in the field and experiments in the laboratory that frost damage in concrete occurs by segregation of ice into layers in the concrete and that the layers exert pressure by growth in a direction opposite to the flow of heat. Collins' explanation is as follows:

> Cooling begins at the exposed surface and extends slowly inwards. When any layer below the surface reaches a sufficiently low temperature, the water in the largest pores begins to freeze and the latent heat given up by it tends to maintain constant temperature at the point of ice formation. The ice crystals so formed are in contact with unfrozen water in the surrounding, smaller pores and, by drawing water from them, the crystals continue to grow.
>
> . . . The force exerted by ice will be perpendicular to the cold surface; and if the concrete is of low strength, a plane of weakness parallel to the cold surface will tend to form at the level at which the ice is forming.
>
> The water drawn in by the growing crystals of ice will come first from the largest unfrozen pores. As these become emptied, the supply will be restricted and the rate of growth of the ice will be checked. The evolution of latent heat will not then be sufficient to maintain the temperature constant at the point of ice-formation and the temperature will begin to fall once more.
>
> As there is then little or no water in the largest pores in the concrete immediately below the first ice layer, freezing will not begin again until either the temperature has dropped sufficiently to freeze the pores that do contain water or a level is reached where the larger pores are not affected by the ice forming above them. The result of this process is that concrete will contain a series of planes of weakness parallel to the surface of cooling. During the subsequent cycles of freezing the ice will again tend to form at the same levels as before, because the pores there have been distended by the previous ice, and the freezing point of the water in them will be higher than in the surrounding concrete.
>
> The damage to the concrete is considered to be caused not so much by the actual increase in volume of the water in the pores on freezing as by the growth of the crystals afterwards and the consequent segregation and concentration of ice into the layers.

*A. R. Collins, *J. Inst. Civil Engrs.* **23**, 29 (1944).

To substantiate this conclusion Collins made experiments on concrete cylinders, of the same kind that Taber made on soils. In these experiments the specimen is cooled at one end only while water, kept above the freezing temperature, is supplied at the opposite end. When using a very low-grade, porous cylinder of concrete, Collins observed the development of a horizontal crack about one inch below the top of the concrete cylinder and considered this to be analogous to the laminations which appeared in a soil cylinder under the same condition of test.

With respect to the effect of air-entraining agents Collins says: "The precise way in which these materials improve durability of concrete is not yet fully understood, but they are believed to act in one of two ways. They may change the pore structure of the concrete by entraining air or by forming bubbles of gas, or they may make the concrete itself water repellent. In either case the normal movement of moisture within the concrete is disturbed or restricted and the ability of the water to form ice layers is thereby apparently reduced."

Collins' observations of the behavior of concrete in the mild climate of England led him to the conclusion that concrete mixes so proportioned as to have a water-cement ratio of less than 0.7 by weight are able to withstand frost action.

DEFENSE OF HYDRAULIC PRESSURE PREMISE

As indicated in the introduction, the writer has no desire to try to refute the Taber-Collins hypothesis*; on the contrary, he feels that it should be kept in mind and that experiments should be made to test the premise on which it is based. If still other hypotheses can be devised, so much the better, as far as progress toward a correct understanding of this question is concerned. However, to justify offering a hypothesis that is in basic disagreement with the ideas set forth by Collins, some discussion of the Taber-Collins hypothesis is advisable.

Although Taber's hypothesis seems highly satisfactory for explaining frost heaving in soils, there are several reasons for believing that it should not be applied to concrete or at least not to concrete frozen under the usual laboratory conditions. Some of these reasons rest on direct evidence; some on inference.

Possibly the most direct evidence against the Taber-Collins hypothesis is found in a paper published by Mattimore.† In this paper data are given showing that in laboratory tests the *lower* the rate of freezing the *smaller* the destructive effect, whereas the Taber-Collins hypothesis requires the opposite result.

*This designation is used for convenience. The writer does not know whether Taber has ever applied his hypothesis to concrete or whether he would endorse such an application.

†Proc. Highway Research Board *16*, 135 (1936).

Inferred evidence against applying the Taber hypothesis to concrete comprises those factors and observations indicating that concrete must freeze as a "closed system," whereas segregation of ice can occur only in "open systems." Taber defined a closed system as " . . . systems from which water could not escape and into which water could not enter,"* Leading to this definition is the observation that when a liquid freezes in a porous system, unfrozen liquid at a higher temperature tends to flow toward the ice that is first formed and thereby to promote the growth of the first-formed ice crystals. When water is available and able to flow freely enough to the point of initial ice formation, ice will not form at a lower level as long as this condition persists. The condition of flowing "freely enough" is one in which the heat given up by the water on freezing is enough to maintain constant temperature.

When the heat is conducted from the region of initial ice formation considerably faster than it can be transferred to that region, then the water in the specimen freezes in place and lenses do not form. Regardless of whether or not outside water is available to enter the specimen, Taber calls the condition first described "freezing in an open system," and the one next described "freezing in a closed system."

Taber observed that with respect to ice segregation, "The chief factors are: size and shape of soil particle, amount of water available, size and percentage of voids, rate of cooling, and surface load or resistance to heaving." These are the factors that determine the permeability of the soil, its thermal conductivity, the amount of latent heat released on freezing, per unit volume of soil, and the mechanical force acting against the expansion of the ice.

Concrete differs widely from soil with respect to some of these properties. Ruettgers, Vidal, and Wing† show the permeability of concrete to range from about 16×10^{-8} ft. per day for concrete having a water-cement ratio of 0.44 (5 gal. per sack) to $14{,}000 \times 10^{-8}$ for w/c = 1.0 (11 gal. per sack). Dimitri Krynine‡ gives the permeability of clay as ranging from 0.48 ft. per day ($48{,}000{,}000 \times 10^{-8}$) to "practically zero." Plummer and Dore** give the permeability of clay as "less than 0.00024 ft. per day" ($24{,}000 \times 10^{-8}$). Taber found that the less permeable clays froze as closed systems, that is, there was no segregation of ice even without a superimposed load. Since Taber did not publish the permeabilities of these clays, it is not certain whether their permeabilities were "practically zero" or somewhat higher. Yet it is clear that all but the very leanest concretes must have what the soil tester would judge by his method of test to be practically zero permeability. Among such concretes the

J. Geol. 38, 303 (1930).
‡*Soil Mechanics* (McGraw-Hill, 1941) p. 60.
†*Ibid.* p. 395, Fig. 8.
**Fred L. Plummer and Stanley M. Dore, *Soil Mechanics and Foundations* (Pitman, 1940) p. 45.

degree of impermeability alone precludes the formation of lenses under the usual rates of cooling. Even for the leanest of practical concrete, which for want of definite data must be assumed to have permeabilities possibly as high as some clays, we must conclude that lens formation is highly improbable because of other factors discussed below.

When ice tends to expand by lens growth, it meets much more resistance in concrete than it does in unloaded soil. The ice first formed in concrete could occupy not over 10 percent of the cross-section. This ice cannot grow to form a lens unless it can first disrupt the capillaries in which it forms. The force required for disruption will depend on the tensile strength of the concrete. As shown earlier, this force could probably be as high as 2500 psi.

In soils the voids run upwards of 35 percent. Therefore, even for the same total force the unit pressure on the ice first formed in soil would be about ⅓ to 1/7 of that in concrete. It is very significant that Taber found that loading a soil often caused it to freeze as a closed system, whereas the same soil formed lenses when frozen unloaded. He says, "A relatively small surface load will entirely prevent frost heaving in an open system if the material is of such texture that only a little segregating ice forms under the most favorable conditions."

Thus consideration of permeability and resistance to expansion makes it appear that if any kind of soil freezes as a closed system, then all concrete should do so except perhaps when the rate of cooling is exceedingly low.

It would seem reasonable, however, to assume that ice segregation takes place in concrete on a submicroscopic scale. As already pointed out, the hardened paste is considered to be composed largely of a porous gel containing a system of capillaries. If the ice forms only in the capillaries, as seems likely, then we may imagine it to be forming in a container having porous, water-laden walls of gel. Hence, the ice could and probably would receive water from the walls as well as from the more open capillaries. The amount of water available from the gel is very limited; nevertheless some expansive force due to this effect is probable. In fact, we could have assumed that the expansion on freezing is due entirely to segregation on this submicroscopic scale but, as pointed out before, such an assumption does not lend itself readily to an explanation of the protective effect of air-filled cavities or the observed effect of increasing the rate of freezing. It would require the assumption that the cavities do not directly relieve the pressure but merely permit the solid material to yield under stress with less disruption. How almost complete immunity to frost action could be brought about through this mechanism is not very clear.

SOME GENERAL OBSERVATIONS

Two corollaries of this hydraulic-pressure hypothesis have been given; another will be mentioned here.

It is apparent that with sound aggregates, and with the concrete not initially saturated, the rate of surface disintegration depends mainly on *the porosity of the paste*. The porosity of the concrete as a whole depends on the porosity of the paste and on the paste content of the concrete. Since paste content is an independent variable, it follows that:

No general correlation between rate of disintegration and porosity of the whole concrete should be expected.

Seeming exceptions to this are the results reported by Hansen* where fairly satisfactory correlation between absorption and durability was found for cores cut from highways and specimens cast in the laboratory. However, the absorptions reported were the quantities of water taken up during two days of soaking, after only two days of drying from a previously soaked condition. Owing to the fact that the water lost during a two-day period of evaporation can be practically all reabsorbed in two or three hours, the values reported by Hansen must have depended on the *rate of evaporation* during the drying period. This, in turn, depended not primarily on the total evaporable water content (total porosity) but very largely on the *permeability* of the concrete. Hence, his correlations were virtually between permeability and rate of disintegration. Since the permeability of such concretes depends predominantly on the porosity of the paste, the correlation was virtually on the basis of paste-porosity.

The hypothesis supports strongly the contention of Scholer† and others to the effect that the degree of saturation (or the "saturation coefficient") is "of utmost importance." In fact we may conclude that the two factors that control the life of a specimen in a freezing and thawing test are its initial saturation coefficient and its permeability, together with these same factors for any unsound aggregate in the concrete. The actual life of the specimen will depend on the conditions that control its absorption of water during the test, as discussed above, assuming that the specimen as a whole was initially below the critical degree of saturation.

TESTING THE HYPOTHESIS

This hypothesis cannot be proved or disproved by data now at hand. To test the hypothesis, quantitative data on such factors as *initial degree of saturation* and *absorptivity* of the test specimens must be known.

*W. C. Hansen, "Uniformity of Cores an Indication of Pavement Quality," *Proc.* 20th Ann. Meeting Highway Res. Board, p. 568 (1940); ACI JOURNAL, Nov. 1942; *Proc.* 105 (1943).

†C. H. Scholer, "Durability of Concrete," Report on Significance of Tests of Concrete and Concrete Aggregates, 2d Ed., p. 29 (1943) A.S.T.M. Publication.

Without such quantitative information, one can use the hypothesis to explain *any* observed result, simply by making appropriate assumptions about the unknown quantities.

For example, more than once it has been observed that prolonging the curing period reduces the resistance of laboratory specimens in a freezing and thawing test. Such a result can be explained as follows: An increase in the amount of hydration has three possible effects:

(1) It reduces permeability.
(2) It reduces the amount of freezable water.
(3) It possibly increases the initial degree of saturation.

According to the hypothesis a reduction in permeability and an increase in degree of saturation are factors increasing the intensity of hydraulic pressure and are therefore detrimental to durability. On the other hand, the reduction in the amount of freezable water is beneficial. To account for the observed effect it is only necessary to assume that the effect of reducing permeability or increasing the initial degree of saturation, or both, outweighed the effect of the reduction in freezable water. To account for an *improvement* in resistance from prolonged curing, which also has been observed, it is only necessary to make the opposite assumption.

The principal incentive for formulating this hypothesis is the hope that those who have the necessary facilities will begin to accumulate such data as are required to support or refute the ideas set forth. This requires new knowledge of the properties of paste and of aggregate, particularly their relative permeabilities. It also requires data on the permeability and absorptivity of the concrete as a whole. It requires new studies of supercooling.

With respect to testing technique it requires close control of freezing and thawing conditions, particularly the rate of cooling, the minimum temperature reached, the amount of ice-coating, if any, and the length of time during which the surfaces remain at 32 F during the thawing period.

It requires an accurate measure of the degree of saturation of each specimen at the beginning of the freezing test and a study of the conditions that control the degree of saturation.

The hypothesis indicates also that entrained air should be more effective the smaller the bubbles. Hence, if there is any way to control bubble-size, it would be advantageous to make the bubbles as small as possible and thus minimize the total volume of air required, for to do so should hold to a minimum any adverse effect of entrained air on

strength. This aspect of the hypothesis calls for laboratory experiments along several lines.

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Discussion of a paper by T. C. Powers:

A Working Hypothesis for Further Studies of Frost Resistance of Concrete*

By RUTH D. TERZAGHI, DOUGLAS McHENRY and H. W. BREWER, A. R. COLLINS and AUTHOR

By RUTH D. TERZAGHI*

SCOPE OF DISCUSSION

In his stimulating article on the frost resistance of concrete, Mr. Powers has described a possible mechanism of the disruption of concrete by freezing. In this discussion an attempt will be made to find out whether this mechanism might lead to the disruption of concrete in the field. According to Mr. Powers' hypothesis, disruption may be due to the development of a high hydrostatic pressure in the pore water of concrete adjacent to the zone of freezing. As Mr. Powers has pointed out, this process can be effective only if the concrete is saturated to a certain minimum depth which he terms the critical depth of saturation. Hence we may test the applicability of this hypothesis to field conditions by estimating this depth for concrete of given properties, exposed to natural winter weather, and comparing our estimate with the depth to which such concrete is likely to be completely saturated under normal conditions. Such an estimate is presented in the section on "Critical Depth of Saturation." The results of the estimate suggest that disruption of lean concrete under natural climatic conditions is due to some process other than that postulated by Mr. Powers.

Another topic with which Mr. Powers' article deals is the classification of the pores which contain freezable water. To the two categories of such pores mentioned by Mr. Powers, the present writer would add a third, consisting of the capillary spaces between aggregate and paste. Evidence for the existence of such spaces is presented under the heading "Locus of Freezable Water."

*ACI JOURNAL, Feb. 1945; *Proceedings* V 41, p. 245.

†Geologist, Winchester, Mass. (now cooperating with her husband, Karl Terzaghi on a textbook on engineering geology).

A part of Mr. Powers' article is devoted to a previously stated hypothesis that ice lenses, analogous to those which develop in some soils and sediments, may form in concrete. His conclusions regarding this hypothesis are based in part on the opinion that such ice lenses do not form in relatively impermeable soils. This opinion is not supported by observations of geologists, which are discussed under the heading "Conditions for the Formation of Ice Lenses in Soils and Sediments."

CRITICAL DEPTH OF SATURATION

The purpose of this section is to evaluate the extent to which Mr. Powers' hypothesis may be applicable to the disruption of concrete by frost under natural climatic conditions. The basis for the evaluation is provided by an estimate of the critical depth of saturation corresponding to the most unfavorable winter conditions likely to be encountered.

For concrete with given properties, exposed to natural winter climate, the critical depth of saturation depends on two quantities. These are the rate of decrease of the air temperature from about 32 F to several degrees below that temperature, and the pore water pressure necessary to produce rupture. The first of these is determined by climatic conditions and is probably never in excess of 5 F per hour. The second quantity, the pore water pressure at rupture, is a function of the tensile strength of the concrete and of the shape of the voids.

Published experimental data* indicate that the shape of the voids of concrete is such that the pore water pressure is transmitted to at least 97 per cent of the cross section. Hence we must conclude that the pore water pressure at rupture is approximately equal to the tensile strength of the concrete. This conclusion is incompatible with Mr. Powers' estimate (page 253) that the pore water pressure must become equal to about five times the tensile strength of the concrete in order to damage it. However, the critical depth of saturation will be determined on the basis of both assumptions.

In the following paragraphs, equations will be derived which express the relation between the critical depth of saturation, the rate of temperature decrease of the surroundings, and the pore water pressure at rupture. Conditions at the time of rupture are illustrated by Fig. A.

The symbols which will be used are as follows:

C = constant of integration.
K = coefficient of permeability of the concrete, in cm per sec.
L = latent heat of water per gram = 80 calories.
n = volume of freezable water per unit of volume of saturated concrete.

*Terzaghi, Karl, "Die wirksame Flächenporosität des Betons," *Zeitschrift des Oesterreichischen Ingenieur- und Architekten-Vereines*, Heft ¼, 1934. See also a forthcoming article by the same author "Stress Conditions for the Failure of Saturated Concrete and Rock," Journal American Society for Testing Materials.

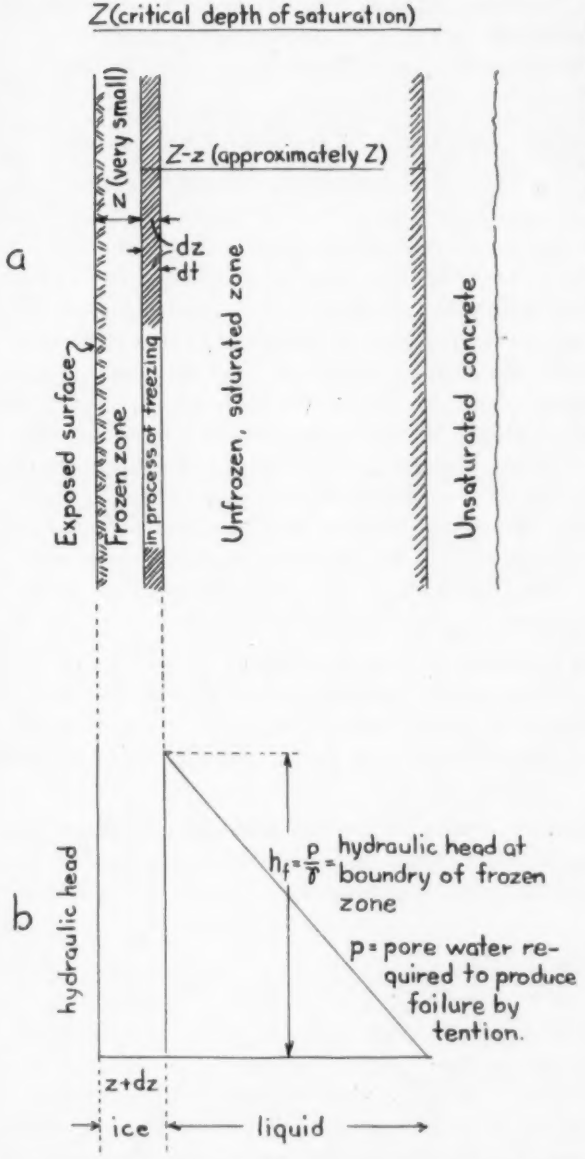

Fig. A—Diagram illustrating conditions during freezing of pore water in saturated zone. a) Cross section through concrete saturated to critical depth Z, b) Hydraulic pressure conditions in unfrozen saturated zone at moment of rupture.

p = hydrostatic pressure in pore water at failure, in gm per sq. cm.
q_c = calories withdrawn from concrete, per unit of area of cross section.
q_w = volume of water displaced, per unit of area of cross section, owing to expansion accompanying freezing.
r = maximum rate of temperature change in degrees centigrade per second under normal climatic conditions, assumed equal to -0.0008 deg. C per sec., corresponding to -3 C (-5 F) per hour.

t = time in seconds
z = depth of frost penetration
Z = critical depth of saturation
γ = unit weight of water
λ = heat conductivity of saturated concrete = 0.003 cal cm^{-1} $degC^{-1}$ sec^{-1}*.
θ_0 = initial temperature, 32 F or slightly below, of concrete and surroundings.

We shall assume that the concrete is completely saturated to depth Z, that it loses heat through one surface only, that the mass has a uniform cross section, and that the concrete and its surroundings are initially at the same temperature θ_0 at or slightly below 32 F which is such that a decrease of temperature will cause freezing. It is assumed that the conditions of exposure are such that the temperature at the surface of the concrete is always equal to the air temperature. We shall also assume that no supercooling takes place, i.e., that the pore water freezes at that temperature at which it would freeze in the presence of an ice crystal. This assumption seems justified, because it is improbable that appreciable supercooling would take place in water-filled pores communicating with free water at the surface of the concrete, which is almost certain to freeze at or near 32 F. Of the several assumptions on which the derivation is based, this is the only one which is a possible source of an underestimate of the rate of freezing.

At time $t = 0$, the air temperature begins to fall at a constant rate r. The water in the pores of the concrete begins to freeze, and the boundary between pore ice and pore water moves into the concrete at a rate dz/dt. At this boundary, the temperature is equal to the freezing point of the pore water θ_0.

If the boundary between ice and water moves through a distance dz, the corresponding volume of water frozen per unit of cross section is $n\,dz$, and the rate at which calories must be withdrawn to freeze this quantity of water is

$$\frac{dq_c}{dt} = L\,n\,\gamma\,\frac{dz}{dt}\ cal\ cm^{-2}\ sec^{-1} \qquad (1)$$

In addition to the latent heat of water, a small quantity of heat dq_c flows out of the concrete on account of the existence of a temperature gradient between the interior of the mass and the outside temperature. A simple computation shows that this quantity is less than 1 per cent of dq_c as long as z does not exceed a few centimeters. Hence it does not require consideration.

The rate at which heat is withdrawn from the zone of freezing, per unit of area of the surface of cooling, is equal to the heat conductivity of the concrete multiplied by the temperature gradient in the concrete

*Knoblauch, Raisch, and Reiher, *Gesundheits Ingenieur*, v. 43, p. 607, 1920; cited in *International Critical Tables* (1927) v. 2, p. 314.

between the zone of freezing and the surface. The temperature in the zone of freezing is equal to the freezing temperature of the pore water, θ_0, which is identical with the temperature of the entire mass of concrete at time $t = 0$. At any time t, the temperature of the surface of the concrete is $\theta_0 + rt$, and the distance between the surface and the zone of freezing (depth of frost penetration) is z. Hence the rate at which calories are withdrawn at time t is

$$\frac{dq_c}{dt} = \frac{\lambda r t}{z} \; cal \; cm^{-2} \; sec^{-1} \quad \ldots\ldots (2)$$

In reality the temperature at depth z is slightly lower than θ_0 when rupture takes place, owing to the fact that the increase in pressure from zero to p causes a slight decrease in the freezing point. Hence our assumption that this temperature is constant and equal to θ_0 results in a slight overestimate of the temperature gradient, of the rate of heat withdrawal, and hence of the rate of frost penetration.

Setting

$$U = \frac{\lambda r}{L n \gamma}$$

and combining equations (1) and (2) we have

$$z \, dz = U \, t \, dt$$

$$\int z \, dz = U \int t \, dt$$

$$\frac{z^2}{2} = \frac{Ut^2}{2} + C$$

When $z = 0, t = 0$. Hence $C = 0$, and

$$z = t \sqrt{U} \quad \ldots\ldots (3)$$

$$\frac{dz}{dt} = \sqrt{U} \quad \ldots\ldots (4)$$

When a volume of water, per unit of cross-section, $n \, dz$, freezes, a volume of water per unit of cross-section

$$dq_w = 0.09 \, n \, dz$$

must be displaced because of the expansion attending freezing. The rate at which it must be displaced is

$$\frac{dq_w}{d \, t} = 0.09 \, n \, \frac{dz}{dt} \; cm \; sec^{-1} \quad \ldots\ldots (5)$$

The quantity dq_w/dt is equal to the coefficient of permeability of the concrete multiplied by the hydraulic gradient. Both theory and experience indicate that rupture (spalling) takes place near the exposed surface. Hence the hydraulic gradient at rupture is approximately $p/\gamma Z$, and the

initial rate at which water is displaced from the zone of freezing to the inner edge of the saturated zone is given by the following equation:

$$\frac{dq_w}{dt} = \frac{Kp}{Z\gamma} \; cm \; sec^{-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Combining equations 4, 5 and 6, we have

$$Z = \frac{Kp}{\gamma \; 0.09 \; n \sqrt{U}} cm \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

U may be evaluated with sufficient accuracy for present purposes from the data given in the list of symbols. Using these values we find

$$U = \frac{3 \times 10^{-8}}{n} \; cm^2 \; sec^{-2}$$

$$\sqrt{U} = \frac{dz}{dt} = 1.73 \times 10^{-4} \sqrt{1/n} \; cm \; sec^{-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The critical depths of saturation for severe winter conditions, calculated by means of equation (7) for the rich and for the lean concrete described by Mr. Powers on page 255, are 0.2 in. and 7 in. respectively, if rupture takes place at a pore water pressure equal to the tensile strength of the concrete. If this pressure must reach a value equal to five times the tensile strength in order to cause appreciable damage, as assumed by Mr. Powers, the corresponding values of the depth of saturation are 1 in. and 35 in. respectively. A depth of saturation of 7 in., even in lean concrete, is probably not common, and it does not seem likely that frost damage to such concrete is limited to those situations where it is saturated to this depth. Hence we must conclude that frost damage to lean concrete exposed to natural climatic conditions is generally due to some process other than that postulated by Mr. Powers.

LOCUS OF FREEZABLE WATER

Mr. Powers has listed two classes of space of capillary dimensions in concrete, likely to contain freezable water. To these the present writer would add a third, consisting of the space between aggregate, particularly the coarse aggregate, and the paste. Evidence of the existence and relative importance of these spaces is found in the condition of some specimens of concrete recently examined by the writer, as well as in the appearance of laboratory specimens of concrete described by Merkle. This evidence is presented in the following paragraphs.

The specimens examined by the writer are drill cores taken from a large mass of concrete two years after casting. The water-cement ratio of the concrete (by weight) was 0.53. The specimens showed no evidence of internal bleeding, of segregation, or of other original defects which

might have resulted in the presence of abnormal cavities in the concrete. One face of the structure is in contact with sea water and one face is exposed to the atmosphere. Sea water enters the concrete under a hydraulic gradient which varies from place to place between about one and about five. Its access to the interior of the structure has been facilitated by the presence of cracks which developed progressively and had become conspicuous two years after casting. In specimens taken from this structure, within a zone whose upper surface slopes away from the sea-face, each of the medium-sized and large pieces of aggregate, with a diameter of about one-quarter inch to three inches, bears on its surface a more or less continuous film of fresh ettringite ($3CaO.3CaSO_4.Al_2O_3.31H_2O$). In most of the specimens there is little or no ettringite on aggregate smaller than about one-quarter inch. The films are commonly a few hundredths of a millimeter (about one-thousandth of an inch) thick; exceptionally they are as thick as one-quarter of a millimeter. The thickness of the film on a given piece of aggregate is uniform. In a few specimens, the surface of coarse aggregate bears a deposit of ettringite which has been in part replaced by calcium carbonate and other substances. In these specimens, the medium-sized and finest aggregate is commonly covered with a deposit of unaltered ettringite.

The present writer's interpretation of these features is based on the following considerations. Although the constituents of the ettringite may have been derived in their entirety from the paste, the actual distribution of readily visible ettringite crystals can be explained only on the assumption that percolating sea water was necessary for their formation. Likewise, the local replacement of ettringite on coarse aggregate by calcium carbonate can be attributed only to the percolation of water (doubtless sea water) containing calcium bicarbonate or carbonic acid in solution, following the deposition of ettringite. We must, therefore, conclude that water has seeped through the concrete along the contact between aggregate and paste. Since the results of this percolation are most conspicuous at the contact between coarse aggregate and paste, the seepage velocity was doubtless most rapid along these contacts.

Evidence of a similar nature was described by G. Merkle*, who found conspicuous deposits of calcium carbonate at the contact between paste and coarse aggregate in laboratory specimens of concrete through which water with a high calcium bicarbonate content had percolated.

Whether a capillary space between aggregate and paste is universally present in concrete, or only under certain conditions, is of course an open question. Wherever such spaces are present, they would doubtless be among the first to be drained under drying conditions and the first to be

*Durchlässigkeit von Beton, Berlin 1927.

refilled when the concrete comes in contact with water. Under certain circumstances, water contained in them might play an important part in the disruption of concrete by freezing.

CONDITIONS FOR THE FORMATION OF ICE LENSES IN SOILS AND UNCONSOLIDATED SEDIMENTS

On the basis of evidence from several sources, Mr. Powers concludes that the formation of ice lenses in concrete, analogous to those formed in soils and sediments, is unlikely. One line of evidence is stated as follows:

"Inferred evidence against applying the Taber hypothesis of ice lense formation comprises those factors and observations indicating that concrete must freeze as a 'closed system,' whereas segregation of ice i.e., formation of ice lenses, can occur only in 'open systems'." (Closed systems are those whose total water content does not change, owing either to the low permeability of the material comprising it or to the inaccessibility of water.) The implication that ice-lense formation is limited to relatively permeable materials is not in agreement with observations by geologists regarding the conditions of ice lense formation in soils and sediments. It would therefore represent a service to all concerned if Mr. Powers would present in detail the observations on which his statement is based.

Observations made by Taber* in the laboratory may be summarized as follows:

Very slight segregation took place in ground quartz with an average particle size between 6 and 10 microns. Precipitated barium sulphate, very uniform in size, having an average particle diameter of about 2 microns and a maximum diameter of about 3 microns, gave well-defined segregation under favorable conditions of cooling, but no segregation under unfavorable conditions. In materials having a particle size of about 1 micron or less, segregation took place readily. Ice layers formed without difficulty in all of the clays investigated by Taber.

Similar observations regarding the influence of grain size were made by Beskow† both in the field and in the laboratory.

In these observations, there is nothing to indicate that low permeability is an obstacle to ice-lense formation. On the contrary, ice lense formation takes place most readily in the relatively impermeable soils. Hence the question as to whether or not ice lenses can or do form in concrete must be answered on the basis of other evidence.

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*Taber, Stephen, "Frost Heaving," The Journal of Geology, vol. 37, 1929, pp. 427-517.

†Beskow, Gunnar, Soil Freezing and Frost Heaving, Swedish Geol. Surv., Series C, No. 375, 26 Yearbook No. 3, Stockholm 1935.

By DOUGLAS McHENRY and H. W. BREWER*

Mr. Powers has attached a very formidable problem with commendable courage and with a degree of scientific detachment which permits no delusions regarding the complexities of the problem or the possibility of resolving its many dilemmas by the application of a single hypothesis. The writers find no fault with the hydraulic pressure hypothesis—nor indeed with any major phase of the paper—provided it is viewed as intended by the author as "a working hypothesis which, together with other hypotheses, may eventually lead to the desired solution." The comments in this discussion, based largely on experimental work in the laboratory of the Bureau of Reclamation, are not intended to simplify the problem, but rather to emphasize its complexities and the need for further study and still other hypotheses. For example, in accounting for the variable effect of increased hydration on durability, the author lists three pertinent effects:

1. It reduces permeability.
2. It reduces the amount of freezable water.
3. It possibly increases the initial degree of saturation.

According to the hypothesis a reduction in permeability and an increase in degree of saturation are detrimental to durability, while a reduction in the amount of freezable water is beneficial. To the above three pertinent effects at least three others might well be added:

4. It reduces the amount of cement available for autogenous healing.
5. It reduces the ability to creep.
6. It increases the tensile strength.

Thus, at least six factors enter into this phase of the problem, and of these 2 and 6 are favorable while 1, 3, 4, and 5 are unfavorable to durability. Even for a particular concrete it appears that the relationship between durability and degree of hydration is not a simple one. The Bureau of Reclamation has conducted tests in which the mixing and initial curing temperatures were 40, 60, 80 and 100 F. These temperatures were maintained for 24 hours, after which the specimens were fog cured at 70 F for 180 days. The number of cycles of freezing and thawing required to produce an expansion of 0.6 percent was 310, 450, 300, 245, respectively. Apparently for this concrete there is a certain optimum curing temperature, and presumably an optimum degree of hydration, to produce maximum durability under the conditions of test employed.

The author did not discuss the healing effect of continued hydration during the freezing and thawing period, but the writers believe that this may often be a significant factor with respect to durability—especially under laboratory conditions in which the tests are started at an early age.

*Bureau of Reclamation, Denver, Colo.

Reagel* has reported tests which show that thawing at 40 F causes more rapid breakdown than thawing at 90 F, and his results have been confirmed by Hornibrook.† It is inferred from the author's discussion of

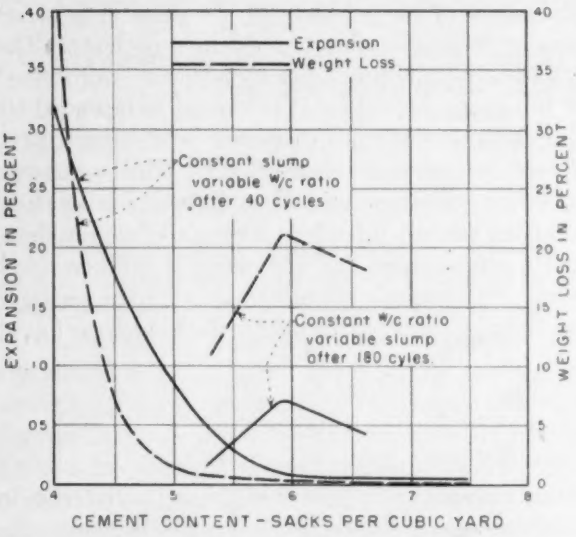

Fig. B—Durability of 3 x 6 in. concrete cylinders.

"Factors governing the amount of water absorbed during thawing" that from consideration of hydraulic pressure alone the reverse should be true because a more rapid melting of the surface ice permits greater absorption during the thawing period. Hornibrook has attributed the greater resistance with a higher thawing temperature to repair by autogenous healing, and it may be that this effect overshadows the effect of the greater absorption discussed by the author.

The results of the freezing and thawing tests on the concrete shown by Fig. 2 are contrary to all test results obtained in this laboratory. Fig. B of this discussion shows expansion and weight loss curves for two typical test series. In both instances the expansion and weight change curves are similar. The writers know of no test results by others which confirm the author's Fig. 2, and it seems that his results must be attributed to some unusual combination of circumstances.

The author mentions fissures under the aggregate particles as one type of cavity which may affect resistance to weathering, but it is not clear whether he attributes to them an increase or a reduction in durability. The writers believe that these fissures contribute to low durability, especially in concrete which bleeds freely. In permeability tests in which

*F. V. Reagel, "Freezing and Thawing Tests of Concrete." Proc. Highway Research Board, v. 20, p. 587 (1940).
†F. B. Hornibrook, Discussion of foregoing reference.

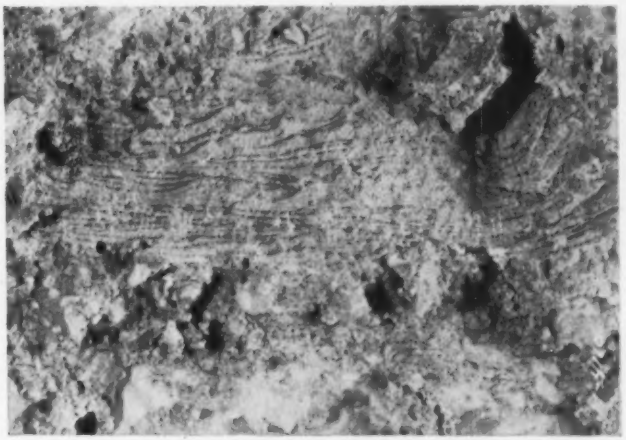

Fig. C—Ice segregation resulting from slow freezing starting when the concrete was 4 hours old — magnified 4½ x

Fig. D—Ice lens formed in soil of permeability corresponding to that of moderately tight concrete—magnified 3 x

dye was used, the greatest concentration of dye was always found in the fissures between the aggregate particles and the paste. Observation of frozen concrete and of disintegrated concrete with the binocular microscope indicates that ice forms readily in such fissures, and that it exerts a disruptive effect by loosening the aggregate particle from its socket. This condition was especially pronounced in cylinders which were frozen slowly from the top downward with the bottom immersed in water. It is considered not unlikely that the concentration of ice in these regions of low tensile strength is closely related to the segregation which the author has discussed under the heading "The Taber-Collins Hypothesis." The writers have not observed any other type of segregation in mature concrete, although distinct lenses have been produced in concrete which was frozen slowly starting when the concrete was only four hours old (Fig. C).

The author remarked on the absence of permeability data on soils which showed ice segregation, but concluded that for concrete "the degree of permeability alone precludes the formation of lenses under the usual rates of cooling." The writers used Taber's technique in testing a cylinder of saturated clay having a permeability coefficient of 1370×10^{-8} feet per day, corresponding to concrete of moderate permeability, and found that a single freezing produced a lens about 1/16 in. thick extending over the entire cross-section of the cylinder (Fig. D). The rate of freezing was slower than the usual rate for freezing and thawing tests, but was not too slow to be representative of some field conditions. The test suggests that the tensile strength, rather than the impermeability, accounts for the usual absence of visible lenses in concrete.

Some test data which have been reported in the literature suggest that the author's figure of 2500 psi as the internal force required to produce disruption of concrete may be too high by a considerable amount (at least, for saturated specimens). Perhaps the best of such data are given by Leliavsky*, who determined the pore pressure required to fracture concrete tubes subjected to axial and circumferential pressures. He concluded that the pore pressure acted over about 92 percent of the cross-sectional area, and that "The fraction of the area of cross-section over which the internal pressure acts can therefore have nothing to do with ordinary porosity." Terzaghi† had previously been led to a similar conclusion by measuring the effect of pore pressure on strains in concrete cylinders. However, the writers do not consider that these tests were conclusive, and believe that more experimental work is needed in this field.

It is believed that much more laboratory work—not haphazard testing, but rather fundamental research—is needed before the mechanism of frost action can be satisfactorily explained. However, the research should not be restricted to the laboratory, for, as demonstrated particularly by recent work of Tremper‡ and Jackson**, valuable clues can be obtained from field inspections. The so-called D-cracks which are often found in concrete structures subjected to freezing and thawing have not been satisfactorily explained. The D-lines which are close to and roughly parallel to edges have been interpreted by some observers as a result of ice segregation.

By A. R. COLLINS§

Mr. Powers' excellent paper has been read with very great interest. He writes with the advantage of having a backing of unpublished data to

*Serge Leliavsky, "Uplift in Dams," *Nature* (London) No. 3770, p. 137 (1942)

†Karl Terzaghi, "Simple Tests to Determine Hydrostatic Uplift," *Eng. News-Record*, v. 116, p. 872 (1936).

‡Bailey Tremper, "The Effect of Alkalies in Portland Cement on the Durability of Concrete," A.C.I. JOURNAL, Nov. 1944, p. 89.

**F. H. Jackson, "Disintegration of Bridge Concrete in the West," Public Roads, v. 24, No. 4, p. 98 (1945).

§Department of Scientific and Industrial Research, Road Research Laboratory, Harmondsworth, West Drayton, Middlesex, England.

support his hypothesis and detailed discussion is, therefore, rather difficult.

Mr. Powers' hypothesis appears to provide a good explanation of most of the data obtained in laboratory tests and perhaps also in some field conditions, but it is felt that there are some circumstances, particularly in regard to concrete pavements, in which ice segregation provides a better explanation of the observed phenomena. This may mean that damage is caused by different mechanisms according to the conditions of freezing and the type of concrete concerned. It appears that Mr. Powers recognizes this possibility when, on page 267, in doubting the validity to the application of the ice segregation theory to concrete, he expressly confines his remarks to freezing in laboratory conditions.

The evidence that suggested ice segregation as the cause of damage in field conditions was obtained in England in early 1942. There was, at that time, an unusually long, severe and continuous frost with little or no intermediate periods of thawing. Throughout the freezing period pavement concrete of poor quality on a certain site appeared to be undamaged, though it was not subjected to traffic. When the final thaw occurred, however, a spectacular failure took place, the concrete being broken up in horizontal layers to its full depth of 6 in. This failure seems to be more easily explained by the formation of ice lenses than by hydraulic pressure.

In laboratory tests in which freezing was applied to one face of the concrete and a water reservoir maintained at the opposite face (conditions which more closely represent practice than the usual freezing test), the total moisture content of the specimen at the end of one period of freezing and before thawing was found to have increased. This suggests a process of ice crystal growth rather than one in which the water is forced away from the cold face. Similar observations have been made on stabilized soil frozen in the same way. There is also direct evidence of ice crystal growth in concrete in the appearance of sizable ice crystals beneath, and supporting, a scale of concrete forced off during freezing*, the volume of ice being much greater than that of any water that could have been driven from the scale by freezing. This phenomenon appears to be similar to the formation of the so-called "fibre ice" under the particles of soil lifted in the same way and reported by Ruckli†.

As Mr. Powers suggests, further tests are clearly necessary to establish the behavior of water in concrete during freezing, but in making these tests we must be sure that a wide range of both laboratory and field conditions are covered. It might then be found that the mechanism of

*A. R. Collins, "The Destruction of Concrete by Frost," *Jour. Inst. C. E.*, 1944, 23(1), 29-41.
†R. Ruckli, "Die Gefährlichkeit des Strassenuntergrundes," *Strasse und Verkehr*, 1943, 29, (19), 311.

failure may vary according to the properties of the concrete and the freezing conditions.

AUTHOR'S CLOSURE

The written discussions submitted by Mrs. Terzaghi, Messrs. McHenry and Brewer and Mr. Collins, together with many oral discussions with Messrs. Pickett and Steinour of this laboratory have been very helpful in further developing, clarifying and modifying the author's hypothesis. This will be apparent in the following discussion of some of the written discussions of the paper.

From the information given by Mrs. Terzaghi and by Messrs. McHenry and Brewer concerning the fraction of cross sectional area contacted by the mobile water in saturated concrete, it appears that the author's computation of the hydraulic pressure required to produce stress equal to the strength of concrete gave a result that is too high. If the area of contact is 100 percent, the tensile stresses developed under the conditions assumed would be five times as high as those estimated by the author, as Mrs. Terzaghi has pointed out. This would seem to increase the force of the author's argument that disruptive hydraulic pressures are likely to develop through the displacement of unfrozen water.

Mrs. Terzaghi's computation indicated that among grades of concrete originally used, the critical depths of saturation range from 0.2 in. to 7 in. for cooling at a "natural" rate. This too might be taken as added support of the hypothesis but for the fact that the basis of the computations is incorrect. For her computations, Mrs. Terzaghi adopted the same simplifying assumptions used by the author for the purpose of illustrating the principles on which the hypothesis is based. Unfortunately, these assumptions are not permissible for a computation such as Mrs. Terzaghi undertook. The author assumed that all the freezable water would freeze at the same time. This would probably be not very far from the truth for small specimens rapidly cooled to a very low temperature.* Mrs. Terzaghi not only adopted this assumption but also the further assumption that all the freezable water would freeze "at or slightly below 32 F." This latter assumption is definitely not acceptable for such a computation, for when concrete is cooled comparatively slowly, there must be a considerable time-lag between the first and final freezing in a given region, if supercooling does not occur.

An example of melting-data for water in a saturated cement paste is:

*Nevertheless, had the significance of progressive freezing been realized at the time (see below), this assumption would not have been made, even though the author's aim was restricted to illustrating the general principle involved.

Temp. deg. C	Amount of Ice, g/g of cement	Amount of Ice as per cent of Amount at − 15 C
0	0	0
−0.25	.045	21
−0.75	.075	36
−1.00	.093	44
−1.50	.109	52
−2.00	.122	59
−2.50	.131	62
−3.0	.137	65
−4.0	.147	70
−6.0	.168	80
−8.0	.181	86
−15.0	.210	100

From this it must be concluded that the amount of water freezing at any one time and place is much less than the total freezable water as defined in the paper.

Further consideration of this phase of the hypothesis gives rise to the following speculation: Since the water may freeze progressively as the temperature falls, the ice content will have a gradient

$$\frac{di}{dz} = \frac{di}{d\theta} \cdot \frac{d\theta}{dz}$$

where $\frac{di}{dz}$ = the ice-content gradient

$\frac{di}{d\theta}$ = the change in ice content per degree drop in temperature

$\frac{d\theta}{dz}$ = the temperature gradient.

This would indicate that the ice that forms at low temperature would form in spaces already partially or wholly surrounded with ice formed earlier when the temperature was higher. The passages would either be much restricted or wholly blocked by the earlier-formed ice and thus the pressures generated by the formation of the ice at low temperature might be much higher than would be estimated from the normal permeability of the concrete. Also it might be of more nearly static nature than was first supposed.

For the sake of simplicity the author assumed that the saturated region of the concrete was fully saturated and that when this region froze, the displaced water flowed across a sharp line of demarcation into a region containing no freezable water. This was pointed out as an oversimplification (see pages 254 and 255) justifiable only for the purpose of providing a simple illustration of the principle on which the hypothesis is based. The oversimplification results in an underestimate of probable hydraulic pressure when applied to the problem attacked by Mrs. Terzaghi. To illustrate this point, we may imagine a case wherein, at saturation, the amount of water freezable at −15 C is n grams per cu. cm of concrete. We will assume that the specimen is saturated to the depth z, that the freezable water content of the rest of the specimen is $0.9n$, and that the mobile water content at saturation is equal to the freezable water content. When the saturated region is frozen to a depth of 1 cm, about $0.1n$ cu. cm of water will be forced into the region already containing $0.9n$ cu. cm of freezable water per cu. cm of concrete. Therefore, for each cu. cm of saturated concrete that is frozen, 1 cu. cm of the partially saturated region would become saturated. Thus, the depth of the saturated zone, under the conditions assumed would increase at the same rate as the rate of freezing.

This too is an oversimplification acceptable only for the purpose of illustration. Experiment shows that, except for very porous concrete, water does not penetrate dry concrete in such a way as to give a sharp demarcation between the saturated and unsaturated zones. Instead, the process resembles diffusion in that a moisture gradient develops that becomes flatter the farther the water moves into the dry interior. This fact and the probable effect of progressive freezing on permeability as discussed above would have to be taken into account to reach even an approximate solution of the problem attempted by Mrs. Terzaghi.

Mrs. Terzaghi feels that lean concrete is not likely to become saturated to a depth of 7 in. and because of this opinion she feels forced to the conclusion that frost damage to lean concrete is generally due to some process other than that postulated by the author. Such a statement indicates that the discussion of the possible role of dynamic pressure in the destruction of laboratory specimens tended to obscure the importance of static pressures in general. The hypothesis is based on the premise that the destruction of concrete is caused by hydraulic pressure generated by the expansion accompanying the freezing of water (see p. 246). This pressure may be either static or dynamic. As pointed out on page 257, failure because of static pressure will occur when escape from a saturated region is impossible, even though the depth of saturation is less than the critical depth.

To illustrate further: It might be surmised from a part of the hypothesis that a 6-in. retaining wall made of concrete having a critical depth of saturation of 7 in. would never be damaged by freezing and thawing. Such a conclusion should not be reached on the basis of the whole hypothesis, however. Failure might occur through unsound aggregate particles or it might occur as follows: Suppose the wall to have become saturated from ground water and rain and that it has become frozen by a gradual drop in temperature without appreciable damage to the wall. If the exposed side then thaws to a depth of, say, 1 in. with perhaps rain or melting snow to keep the surface saturated, a subsequent drop in temperature could cause damage by trapping unfrozen water between the ice formed near the exterior and the ice still in the interior. Under such a condition the failure would be caused by a static pressure.* To be considered also is the probable effect of progressive freezing on permeability, duscussed above. Probably the critical depth of saturation in lean concrete is much less than was indicated by the result of Mrs. Terzaghi's computation.

Regarding the probability of ice-segregation in concrete: As Mr. Collins observed, most of the discussion in the paper pertained to the probability of lens-formation under laboratory-test conditions. The author agrees that there must be some condition of cooling that could produce ice-segregation if the condition is maintained long enough, and if the concrete next to the lens is kept virtually saturated while the lens grows. The question is whether such a condition is ever found in the field, or at least whether it is found frequently enough to be considered the usual mode of concrete destruction by frost action. The author still sees little possibility that segregation can occur, even in the field, because of

(1) the relatively low freezable water content,
(2) the smallness of the portion of the total freezable water that is freezable near the normal freezing point,
(3) the "self-desiccation" caused by cement hydration,
(4) the relatively low permeability, and
(5) the rigidity of the concrete.

I must disagree with Mrs. Terzaghi's statement that there is nothing in Taber's observations to indicate that low permeability is an obstacle to ice-lens formation. In his paper "Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements" *Public Roads* V. 11, 113 (1930)), Taber states: "In laboratory tests, soils that are very impermeable because of high colloid content behave essentially as closed systems, the amount of heaving being the same whether the containers

*This explanation was given by Kreüger (ibid.) for certain types of masonry failures in buildings.

are sealed at the bottom or whether they are perforated and stand in water. The surface uplift in both cases is equal to the change in volume of the water frozen. . . . . The only soils thus far tested that behave on freezing like closed systems, because of impermeability, are certain muck and gumbo soils, and soils containing bentonite." In the *Jl. of Geology*, V. 38, page 306, Taber says, "As the resistance to movement of ground water increases, an open system tends to grade into a closed system." In the laboratory Taber succeeded in producing ice segregation in mixtures containing bentonite, but in order to do this he found it necessary to "mix the bentonite thoroughly with water before placing it in cartons." The point is that when a system contains a large amount of readily freezable water, as did Taber's laboratory preparations containing bentonite, and probably the saturated clay shown in McHenry and Brewer's Fig. D, perhaps 30 to 40 per cent of the over-all volume, lenses could form under ordinary cooling conditions by extracting water from the material immediately adjacent to the points of lens formation. Also, in such specimens, most of the freezable water is freezable near the normal freezing point. Permeability becomes a deciding factor when the material close to the zone of freezing does not contain enough readily freezable water to produce a visible lens.

The part that permeability plays in the phenomenon of ice segregation can readily be illustrated by means of Fig. 1 of the paper. For the present purpose imagine the concrete represented in Fig. 1 to be saturated throughout and in communication on the right-hand side with a supply of water. Heat is being extracted through the layer of ice at the left. Under certain conditions of cooling the water in the concrete will not freeze; instead, the thickness of the ice coating will increase by extracting water from the concrete.

The conditions under which this phenomenon will occur are those in which heat is removed at a rate just sufficient to freeze the water as it is extracted from the concrete. If heat is removed at a greater rate, the temperature will fall and the zone of freezing will move into the concrete. Since the rate of flow of water from the concrete depends on permeability the permissible rate of heat removal for ice lens formation obviously depends on the permeability of the concrete.

It is true that segregation occurs only in granular materials of relatively high specific surface, as Mr. Terzaghi pointed out. But this fact is not incompatible with the conclusion reached above.

This discussion is intended to show the conditions that must be maintained while a given ice-lens grows. It does not deal with the conditions that must be met for an ice-lens to start within a rigid material like concrete. As Messrs. McHenry and Brewer point out, the tensile

strength of the concrete must be overcome before a lens can start, unless it starts from water in a macroscopic cavity and even then it cannot grow beyond the cavity without rupturing the concrete.

Mrs. Terzaghi and Messrs. McHenry and Brewer express similar views concerning spaces between the aggregate and the paste. These spaces can undoubtedly be sources of disruptive hydraulic pressure, or perhaps of the growth of ice lenses, but only after the concrete has been subjected to prolonged external hydraulic pressure of considerable intensity. The condition of specimens after permeability tests, as mentioned by the discussers, shows that such spaces may become filled, at least partly. Also, this is indicated by pressure cells embedded near the upstream face and near the base of a high dam. But experience shows that in concrete of good quality, under conditions where external hydrostatic pressure is low or absent, the fissures under the aggregate become empty and the original air-filled cavities remain empty. Moreover, the capillary system of the paste itself becomes partly empty. When the primary cause of water-movement into such concrete is capillary absorption, water would move into such spaces last, if at all. Hence, fissures and other cavities in laboratory specimens, pavement slabs and other members not subjected to prolonged high pressure, should, according to the hypothesis, be beneficial to frost resistance. The known beneficial effect of entrained air is strong support to this conclusion. Also, we have data indicating that the under-aggregate fissures increase the frost resistance of concrete specimens soaked in the usual way before freezing. When powdered minerals are added to lean concrete or mortar, both the air-voids and the fissures are reduced. Such effects are accompanied by *decreased* frost resistance even though strength is usually increased.

Messrs. McHenry and Brewer note that no satisfactory explanation of D-cracking other than ice segregation has been offered. Such cracks, running parallel to each other and to the edge of a slab, can be explained by the hydraulic-pressure hypothesis.

In wet weather, water accumulates in transverse cracks and joints and in the shoulder next to the edge of the pavement. Usually water is retained in these regions long after the surface of the slab becomes dry. Consequently, the margins of the slab tend to become and remain saturated. When freezing of the saturated margins occurs, pressure is developed in them tending to expand and rupture these saturated margins. However, expansion in a direction parallel to the edge is restrained by the unsaturated portion of the slab and therefore cracks due to freezing never run perpendicular to an edge as shrinkage cracks do. The D-crack first formed readily carries water and thus aids in increasing the width of the saturated margin and thereby promotes the formation of additional parallel cracks when freezing again occurs.

A proper rate of cooling could produce surface scaling by means of ice-segregation, but it appears more probable that scaling occurs when saturated surface layers are literally pushed up by the hydraulic pressure.

The writer believes that leaching of lime and alkalies from the margins of a concrete slab is a factor hastening the process and possibly is essential to it. Leaching coarsens the texture of the paste and thereby increases its capacity for freezable water. Such an increase in freezable water increases the amount of water that must be forced into the unleached and only partially saturated region in a given length of time and would thus result in increased hydraulic pressure.

Regarding Fig. 2 on page 258: This laboratory has data, like those mentioned by Messrs. McHenry and Brewer, that apparently contradict the results given in Fig. 2. Yet we can find no basis for discarding the results shown in Fig. 2; they, as well as different results, must eventually be explained. Possibly the peculiar combination of size of specimen, amount of water around the specimens, rate of freezing and manner of thawing, and the treatment of the specimens before the cycles began would have to be duplicated exactly to get the same results. Such apparent discrepancies between the results from different laboratories, and indeed from one laboratory, show the need for establishing a test procedure that is based on a better understanding of the mechanism of frost action than we now have.

In the original paper the writer neglected to mention recent papers by H. L. Kennedy.* Some of the basic ideas on the mechanism of frost resistance given in my paper are similar to those published by Mr. Kennedy.

*H. L. Kennedy, ACI JOURNAL, June 1944; V. 40, p. 515.
H. L. Kennedy, Proc. A.S.T.M. V. 44, 821 (1944).

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Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars

REPORT OF ACI COMMITTEE 208, BOND STRESS

and an introductory statement by the chairman. The report proper, beginning p. 278, represents substantial agreement in Committee 208 and was approved by the ACI Standards Committee for publication as information in a preliminary period of discussion and trial. The "foreword" and preliminary discussion were prepared after the committee vote was taken and do not necessarily reflect accurately the points of view of individual committee members.

H. J. GILKEY, Chairman

GEORGE C. ERNST
ROBERT E. GLOVER
H. F. GONNERMAN
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FOREWORD

The Proposed Test Procedure which constitutes the Committee's report is presented as information to elicit constructive criticism. The evolution of the test procedure is the culmination of a series of factors and developments, an understanding of which is essential to its accurate appraisal.

Early in 1944 the American Iron and Steel Institute, through its Research Engineer, R. R. Zipprodt, sought the assistance of the American Concrete Institute through its committee 208 on Bond Stress in the evolution of a test procedure designed to supply valid comparisons of bond effectiveness for different types of reinforcing bars. Within the A.I.S.I. whose member companies manufacture a high percentage of the steel reinforcement used in this country, there had developed a general recognition that many if not all of the currently used types of deformed bars are less effective in developing bond than they ought to be and that, through well designed elimination tests, it should be possible to reduce the twenty or more commercially manufactured bars to a few types all of which would perform as well or better than the best of the bars now on the market.

Recognizing that progress along these lines would be clearly of benefit to the users of concrete and to the concrete industry, the ACI indicated its readiness to cooperate fully with the A.I.S.I. Under the chairmanship of Albert Smith a sub-committee of Committee 208, consisting of H. F. Gonnerman, G. A. Maney and F. E. Richart evolved the test procedure outlined herein which now has the approval of Committee 208 for publication as information.

It is probable that this procedure, or some modification thereof, will eventually be submitted to the ACI Standards Committee with a view to adoption as a Standard of the Institute, and the form of presentation chosen anticipates this possibility.

In spite of what to many appears to be a fairly extensive back-log of literature on bond and bond tests, the gaps in our knowledge are numerous and wide. As is always true where conclusive factual evidence is lacking many pronounced divergent opinions exist. The committee recognizes at the outset the futility of any expectation that the procedure here recommended will receive the approval of all qualified critics, but it does seem desirable, nevertheless, to attempt to anticipate and answer in advance (insofar as the Committee or its chairman has explanations to offer) some of the criticisms or questions which are almost certain to be raised. The following straw-man questions are propounded as a basis for anticipatory discussion.

1. Specifically, what is the primary objective of these tests?

2. Why not a much simpler test; doesn't the pullout test give dependable indications?

3. Why such an elaborate, complete and costly a layout of forms and equipment; even if such a pretentious program is justified, most laboratories could improvise with their existing equipment to approximate the equivalent of many of the items specified?

4. Why use arbitrary nominal areas?

5. Why not include provisions for investigating relative effectiveness of hooks?

6. These short, deep beams are not at all representative of the great majority of reinforced concrete beams; why not attempt more nearly to simulate practical conditions?

7. Why the inverted series AA, BB, etc., (tensile steel near the top of the beam as cast)?

DISCUSSION OF QUESTIONS

1. As previously indicated, this test procedure is designed primarily as a basis for comparing the effectiveness of different types of lugs and surface textures on deformed bars with a view to developing a few types

of improved bars, eliminating inferior bars altogether or even to achieving the ultimate objective of the industry to develop one standard deformed bar. While this explains the origin of the project it is expected that the procedure outlined will also be used as a basis for other fundamental bond studies of which several stand high on the priority list of concrete question marks.

2. Admittedly a pullout test technique would be much simpler than the proposed tests on beam specimens. On the basis of careful consideration the subcommittee was forced to decide, however, that available evidence does not justify an assumption that the bond developed by a bar (especially a deformed bar) in a pullout specimen necessarily equals or even parallels the bond action of the same bar functioning as tensile reinforcement in a beam. Among the more obvious differences are the following:

(a) For the pullout specimen the steel is surrounded, during test, by concrete in compression whereas in a beam the surrounding concrete is in tension. While, on the basis of limited evidence secured from tests on plain bars* the chairman questions the pertinency of this difference, it has significance in the minds of many, and for such persons evidence from pullout specimens cannot be convincing.

(b) In the pullout specimen the bar is surrounded by a block of plain concrete or, as sometimes in the case of deformed bars, by a spirally reinforced block. Neither of these conditions is closely similar to that of the bar in a beam where stirrups offer some lateral restraint against splitting but nothing resembling that offered by a spiral.

(c) The beam is cast horizontally; unreinforced pullout specimens can also be cast horizontally but it would be difficult to employ horizontal casting for spirally reinforced pullout specimens. As demonstrated by Menzel† the orientation of deformed bars at casting has a pronounced influence upon the bond developed.

From the foregoing illustrations it is evident that there are notable and probably significant differences between pullout and beam specimens, especially where deformed bars are being compared; that incontestable evidence on the relative behavior of deformed bars in beams can, at the present time, be secured only by testing such bars in beam specimens. As is pointed out in the discussion of Question 6, the test proposed does, in certain essential respects, resemble a pullout test (more closely, perhaps, than some will deem desirable).

While the procedure here outlined includes no provision for parallel pullout tests (since digressions must be severely limited) one of its adapta-

*Iowa State College Engineering Experiment Station Bulletin 147, Fig. 51, p. 76.

†"Some Factors Influencing Results of Pull-Out Bond Tests," ACI JOURNAL, June 1939; *Proceedings* V. 35, p. 517 (See Fig. 14, p. 539)

tions is likely to be one or more investigations designed to throw added light on the extent to which results from pullout tests can be carried validly over to the bond phenomenon as applied to beams.

3. The test layout is for a fundamental investigation of such importance and magnitude as to justify unusual precautions to exclude stray variables. It is expected that many of the current types of bars would be excluded by preliminary and simpler tests (perhaps some form of pullout test or a much simpler or cruder beam or beam type test) and that only a few bar types would finally be tested in a few of the country's better equipped laboratories. Duplicate or repeat series would be run only as new types of bars were developed and needed evaluation, or at intervals of a few years, perhaps, to verify the constancy of bars previously selected as among the best. It is expected that a sponsoring organization of manufacturers of bars, will fabricate, supply and continue to own the necessary testing equipment, simply lending it to any laboratory which might be engaged for the time being in conducting tests for which such equipment was needed.

4. This is somewhat of a moot question a few of the pros and cons of which are already a matter of printed record.* Designs are made on the basis of nominal rather than of average or minimum areas of cross-section. Since bars are purchased by weight, a consistent tonnage overrun near the upper limit of the specified tolerance, places that bar on the defensive in comparison with its fellows. This item, becomes one of the factors to be pitted by the manufacturers against bond performance in deciding upon the bars to be standardized; it is irrelevant with regard to the measurement of bond performance. Anyone interested in supplementary weight-area relationships can determine them independently at any time by any desired means.

5. Hooks are important but the present investigation is to compare different types of straight bars. Hooks constitute an entirely different aspect of the bond problem, quite without the scope here intended. The technique proposed might well be adapted to basic investigations of hook action.

6. These beams are designed for failure by bond; the shears are therefore, very high in relation to bending, so high as not to be complicated greatly by the other stresses normally present in a beam. The specimens are comparable to foundation slabs and reinforced lintels; they are, in fact, virtually horizontal pullout specimens with length of embedment extending from load point to end of span, the main essential difference being that the concrete around the bar is in tension instead of compression.

7. A major objective for such an investigation as that here outlined is to discover and catalogue the differences between bars with regard to

*Proceedings A.S.T.M., V. 34, Part 1, P. 84 (1934)

water gain and settlement of the concrete beneath the bars. As tests such as some of the unpublished work of Menzel (Portland Cement Association Laboratory) have strikingly demonstrated, the extent of water-gain and settlement is a function of depth of concrete beneath the bar and the inverted series is introduced to secure evidence on how these factors affect bond for different types of bars. With other factors equal, the best deformed bar will be that for which increased settlement and water gain produce the smallest decrease in the bond developed.

ACKNOWLEDGMENTS

The Committee is indebted to D. E. Parsons and Prof. C. J. Posey for constructive written suggestions and to Carl A. Menzel for a great amount of active cooperation and help, especially in the design of details and preparation of the drawings. The chairman is indebted to his colleague, Prof. S. J. Chamberlin for counsel and advice.

H. J. Gilkey, Chairman Committee 208.

The Committee's Report:

PROPOSED TEST PROCEDURE TO DETERMINE RELATIVE BOND VALUE OF REINFORCING BARS*

Beam specimens

1. There shall be eighteen beam specimens, the details of which are shown on the accompanying Drawings 1 and 2.†

2. The nine A, B and C specimens (3 for each length of embedment) shall be cast erect as shown. The nine AA, BB and CC specimens shall be cast inverted.

Concrete

3. The concrete shall have a slump of 5 to 6 in. in the standard slump test, and an ultimate strength of 3,500 psi. (plus or minus 300 psi.) at age of twenty-eight days. Preliminary tests shall be made to determine the proportions of the prescribed materials required to develop this ultimate strength. The cement shall be equal parts by weight of four standard brands of Type 1 portland cement. The fine aggregate shall be natural sand well graded from 0 to No. 4 sieve. The coarse aggregate shall be gravel, all of which shall pass either a sieve having $\frac{3}{4}$-in. square openings or a screen with 1-in. diameter openings. All materials shall conform to A.S.T.M. Standards.

Bars

4. The deformed or plain bars to be tested for bond shall be nominally $\frac{7}{8}$-in. in diameter. The weight shall be measured and reported in lb. per ft. to the nearest 0.01 lb. In the interest of uniformity and simplicity, values for tensile stress, yield point, bond stress and modulus of elasticity will be calculated on the basis of the cross-sectional area and the perimeter listed in handbooks for a plain round bar of the same nominal diameter. Specifically the cross-sectional area of a $\frac{7}{8}$-in. deformed or plain bar (meeting A.S.T.M. requirements for weight tolerances) will be taken as 0.60 sq. in and the perimeter as 2.75 in.

5. The pull at yield point and the relation between pull on bar and elongation shall be determined from several typical samples of each design of bar. This relation will serve to indicate the pull on bar at various stages of the test from the strain measurements to be made at the sides of the bar at notches cast in the beam specimens.

*Released by the ACI Standards Committee Dec. 30, 1944 for publication and discussion.

†The 11 drawings p. 282 to 292 which constitute an important part of the "proposed test procedure" may be obtained from the American Concrete Institute in blueprints, or possibly in black and white lithoprint reproductions—a full set, postage paid, for $2.25. Whether in blueprint (the larger sheets 22 by 35 inches) or in lithoprints (11 by 17 inches) will be the choice of ACI, not of the reader, depending on the extent of the demand evident in orders received by the ACI prior to March 10, 1945.

6. The bars submitted for test shall be representative of the product of the mill which submits them, and shall not be given any treatment to produce an abnormal surface. The surface of all bars, however, shall be thoroughly cleaned with naphtha or carbon tetrachloride just before insertion in mold to remove all oil or grease and any other foreign matter. If, in any case there is a question as to whether the bars submitted are truly representative, new samples shall be selected for the tests by an experienced disinterested inspector.

Molds and assemblies

7. The mold shall be of metal, and shall be coated to prevent adhesion to the concrete and sealed to prevent leakage. The mold assembly is shown in Drawing 4 and the exterior parts of mold are detailed in Drawing 5. Two notches 6 in. long and 3 in. high shall be cast in all beam specimens, as shown in Drawings 1, 3, 4 and the insert for these notches shall be as detailed in Drawing 6.

8. Details of the main reinforcing bar and stirrup assembly ready for installing in the mold are shown in Drawings 10 and 11.

9. The $\frac{7}{8}$-in. bar shall be so placed in the mold that the parting line of its deformation pattern (that is, the parting line of upper and lower halves of bar as rolled) shall come at the sides, see detail, Drawing 4. The 3-in. extensions of the bar beyond the center lines of the reactions shall be covered with a rubber sleeve so as to insure no bond. The stirrups shall be $\frac{1}{2}$-in. round deformed bars, furnished by the maker of the $\frac{7}{8}$-in. bars to be tested, and shall be welded into loops as shown in Drawing 11. All steel reinforcement shall be rigidly held in place in the mold (see Drawing 4).

10. Within the length of each notch, two yokes, similar to those shown in Drawing 3 and detailed in Drawing 7 shall be clamped into carefully drilled contact points on 5-in. centers on each side of the bar. These points shall be so located (by means of the jig detailed in Drawing 10) as to make the two yokes as nearly parallel as is practicable. The pull or the fibre stress in the bar shall be determined from the average of the change in distance between the two yokes.

Casting, removal of forms, curing

11. The three identical specimens representing one length of embedment of a given bar shall be made on different days in order to average minor differences in the mix and other variables. The concrete shall be deposited in two layers and each layer shall be sliced and rodded to insure complete contact with the steel reinforcement. Immediately after placing, the concrete should be struck off level with the top edge of the mold with a minimum of disturbance. The specimens must not be exposed to any

vibrations or other disturbances during the first twenty-four hours after casting.

12. At least three standard 6 by 12-in. concrete cylinders for compression tests shall be made from the various batches of concrete mixed for each beam, one cylinder from any one batch.

13. After preliminary curing in the mold for 5 days and removal of molds as indicated below, the beam specimens and cylinders shall be stored in a standard fog room at a temperature of 65 to 75 F. and tested moist at age of twenty-eight days. While in the molds the beams and cylinders shall be protected against loss of moisture from the concrete.

14. In order to avoid damaging the beam specimens the sides and ends of the molds and the tapered inserts shall not be removed earlier than 5 days after placing the concrete. Care shall be exercised to avoid damaging the green specimens during removal to moist room. To this end it is recommended that the beams be placed on stiff wooden planks or equivalent supports during trucking operations. The pieces forming the notches in the beam are to be withdrawn at the end of the 5-day period in the manner shown in Drawing 6.

Testing

15. The sides of the specimen shall be whitewashed about 2 hours before test and allowed to dry so that the sequence and development of cracks may be easily marked by pencil lines which will be visible on any photographs taken after test.

16. Loads shall be applied through rollers or rockers. The beams shall rest on rocker or roller supports to avoid restraint to elongation of bottom fibres of beam as load is applied. The apparatus for supporting and applying load to beam specimen is shown in Drawing 3 and detailed in Drawing 9.

17. A reference steel angle clip shall be screwed to the concrete of the beam sides as near as is convenient to the notch and on the reaction side of the load as shown in Drawing 3 and detailed in Drawing 7. Between this angle and the nearer extensometer yoke the slip of bar immediately under the load shall be measured. This slip will be designated the slip at "loaded" end of bar.

18. The slip of bar at end of beam shall be measured in the manner shown in Drawing 3 with the gage detailed in Drawing 7. This slip will be designated as the slip at "free" end of bar.

19. The deflection at center of the beam shall be measured in the manner shown in Drawing 3 with the apparatus detailed in Drawing 8.

20. The rate of load application during the beam tests shall not be greater than 2000 lb. per minute nor greater than is consistent with securing reliable test data. Readings may be secured for continuous

application of load if available manpower permits. With a shortage of observers it may be preferable either to stop the machine for securing readings at each increment of load or not to stop the machine but to read only half the gages as a given load is passed; reading the other half of the gages at the next increment. When readings are staggered in this manner the indications of corresponding pairs of gages on opposite sides of the beam should be noted simultaneously as in the case of gages indicating slip and strain at a given notch.

21. Load shall be applied to failure, and the report shall state whether failure of the beam was due to failure in bond, diagonal tension, failure of bottom bar in tension or of top concrete in compression, or by splitting of concrete as a result of failure in bond. Cracks due to settlement of concrete will normally occur in the top surface of the beam specimens AA, BB and CC cast inverted. These should be noted and diagrammed for later reference.

22. Readings of slip at both free and loaded ends of the bar, of pull in the bar, and of deflection of the beam at center shall be taken in such number (preferably not less than fifteen) as to give smooth curves in graphical representations of the test data.

Report

23. Curves showing slip of bar shall have the amounts of slip as abscissae, and the beam loads, pulls or fibre stresses in bar, unit bond stresses and values of $\frac{u}{f'_c}$ as ordinates. The unit bond stress, u, is the value obtained by dividing the indicated pull on the bar in pounds (from strain measurements) by the area in bond. The area in bond is obtained by multiplying the nominal perimeter of 2.75 in. for the $\frac{7}{8}$-in. diameter bar (see Par. 4) by the effective length of embedment (8, 12 or 16 in.). Load-deflection curves shall show the amount of deflection as abscissae and beam load as ordinates.

24. The report shall contain a detailed description of the testing set-up, and of all devices and measuring instruments employed.

Discussion of this paper should reach the ACI Secretary, in triplicate, by April 1, 1945, for publication in the JOURNAL for June 1945.

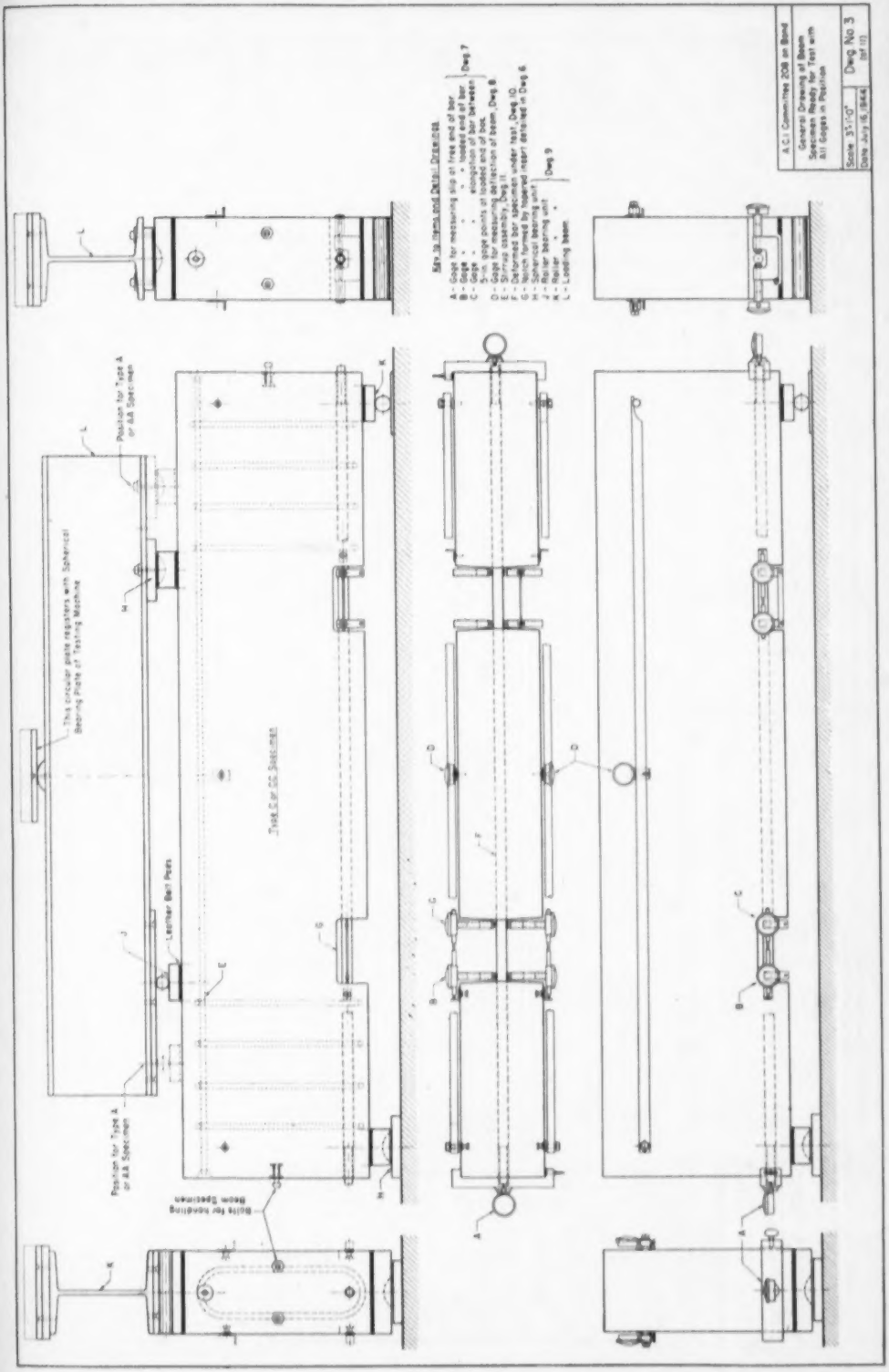
[See drawings 1-11 on the 11 pages following: p. 282 to 292.]

72" Span
Make 3 A
Make 3 AA
Make 3 B
Make 3 BB
Make 3 C
Make 3 CC
All Stirrups are Welded Loops of ¼" rd. Deformed Bars
18 Test Specimens
Spec. A,B,C are cast erect as shown
Spec. AA,BB,CC are cast inverted
Sec. X-X
Sec. Y-Y
A.C.I. Committee 208 on Bond
Beam Specimens for Determining the "Relative Bond Value" of Reinforcing Bars with Various Types of Deformations
Scale: 1"=1'-0"
Date: July 11, 1944
Dwg. No. 1
(of 11)

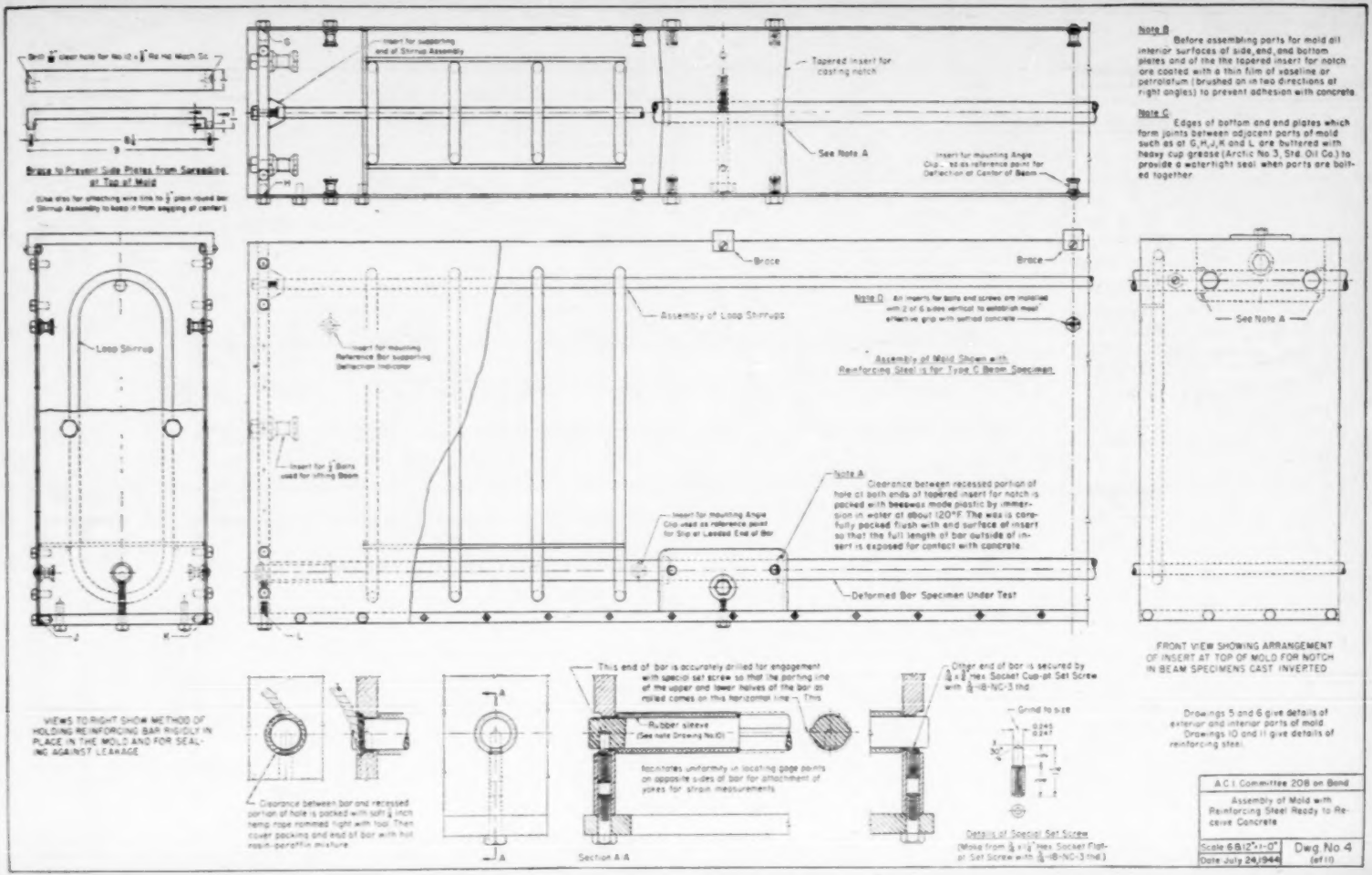
Pull
Pull
Beam cast erect
Beam cast inverted
(tested erect)
Reference Points
A - Slip at free end of bar
B - Slip at loaded end of bar
C - Elongation of bar between 5" gage points at loaded end
D - Deflection at center of beam near neutral surface
P - Applied load
R - Reaction to applied load
A.C.I. Committee 208 on Bond
Diagrams Showing Reference Points for Measurements on Beam Specimens
Scale: 1"=1'-0"
Date: July 13, 1944
Dwg. No. 2
(of 11)

Dwg. 1—Beam specimens for determining the "relative bond value" of reinforcing bars with various types of deformations

Dwg. 2—Diagrams showing reference points for measurements on beam specimens


A.C.I. Committee 208 on Bond
General Drawing of Beam Specimen Ready for Test with All Gages in Position
Scale 3"=1'-0"
Date July 15, 1944
Dwg No 3
(ref 11)
Key to Items and Detail Drawings
A - Gage for measuring slip at free end of bar
B - Gage " " " loaded end of bar
C - Gage " " " " " " between Dwg 7
D - Gage for measuring deflection of beam, Dwg 8
E - Spirrid assembly, Dwg 11
F - Deformed bar specimen under test, Dwg 10
G - Notch formed by fastened paper detailed in Dwg 6
H - Roller bearing unit Dwg 9
J - Roller "
K - Roller "
L - Loading beam
Position for Type A or AA Specimen
This circular plate registers with Spherical Bearing Plate of Testing Machine
Test Bar or Specimen
Leather Belt Path
Bolts for Handling Beam Specimen

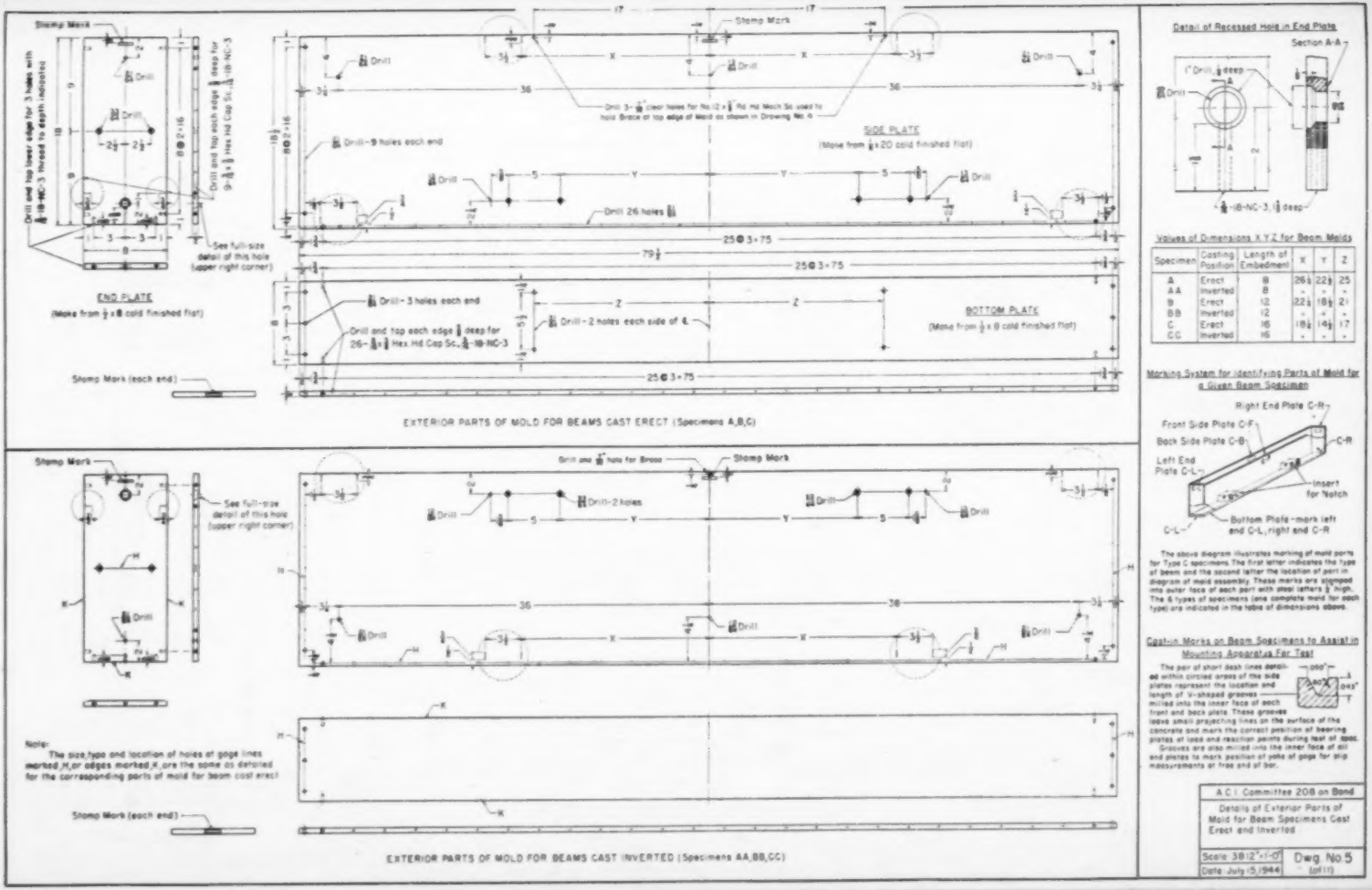
Dwg. 3—General drawing of beam specimen ready for test with all gages in position


Brace to Prevent Side Plates from Spreading at Top of Mold
Insert for supporting end of Stirrup Assembly
Tapered Insert for casting notch
See Note A
Insert for mounting Angle Clip... as reference point for Deflection or Center of Beam
Note B
Before assembling parts for mold all interior surfaces of side, end, and bottom plates and of the the tapered insert for notch are coated with a thin film of vaseline or petrolatum (brushed on in two directions at right angles) to prevent adhesion with concrete.
Note C
Edges of bottom and end plates which form joints between adjacent parts of mold such as at G, H, J, K and L are buttered with heavy cup grease (Arctic No. 3, Std. Oil Co.) to provide a watertight seal when parts are bolted together
Loop Stirrup
Brace
Brace
Assembly of Loop Stirrups
Note D An inserts for bolts and screws are installed with 2 of 6 sides vertical to establish most effective grip with setting concrete
Assembly of Mold Shown with Reinforcing Steel is for Type C Beam Specimen
Insert for mounting Reference Bar supporting Deflection Indicator
Insert for 1 Bolts used for lifting Beam
Note A
Clearance between recessed portion of hole at both ends of tapered insert for notch is packed with beeswax made plastic by immersion in water at about 120°F. The wax is carefully packed flush with end surface of insert so that the full length of bar outside of insert is exposed for contact with concrete.
Insert for mounting Angle Clip used as reference point for Slip at Loaded End of Bar
Deformed Bar Specimen Under Test
See Note A
FRONT VIEW SHOWING ARRANGEMENT OF INSERT AT TOP OF MOLD FOR NOTCH IN BEAM SPECIMENS CAST INVERTED
VIEWS TO RIGHT SHOW METHOD OF HOLDING REINFORCING BAR RIGIDLY IN PLACE IN THE MOLD AND FOR SEALING AGAINST LEAKAGE
Clearance between bar and recessed portion of hole is packed with soft jute hemp rope rammed right with tool. Then cover packing and end of bar with hot rosin-paraffin mixture.
Section A-A
This end of bar is accurately drilled for engagement with special set screw so that the parting line of the upper and lower halves of the bar as rolled comes on this horizontal line. This facilitates uniformity in locating gage points on opposite sides of bar for attachment of yokes for strain measurements.
Rubber sleeve (See note Drawing No 10)
Other end of bar is secured by ... Socket Cup-pt Set Screw with ...-18-NC-3 thd
Grind to size
Details of Special Set Screw
Drawings 5 and 6 give details of exterior and interior parts of mold. Drawings 10 and 11 give details of reinforcing steel.
A.C.I. Committee 208 on Bond
Assembly of Mold with Reinforcing Steel Ready to Receive Concrete
Scale 6 8 12" = 1'-0"
Date July 24, 1944
Dwg No 4 (of 11)

Dwg. 4—Assembly of mold with reinforcing steel ready to receive concrete

(284)

Dwg. 5—Details of exterior parts of mold for beam specimens cast erect and inverted


Stamp Mark
Drill
Drill and tap lower edge for 3 holes with ⅜-16-NC-3 thread to depth indicated
Drill and tap each edge ⅜ deep for 9-⅜-16 Hex Hd Cap Sc. ⅜-16-NC-3
See full-size detail of this hole (upper right corner)
END PLATE
(Make from ½ x 8 cold finished flat)
Drill 3-⅞ clear holes for No. 12 x ½ Rd. Hd. Mach Sc. used to hold Brace at top edge of Mold as shown in Drawing No. 4
SIDE PLATE
(Make from ¼ x 20 cold finished flat)
Drill-9 holes each end
Drill 26 holes
25@3=75
79½
Drill-3 holes each end
Drill-2 holes each side of ℄
Drill and tap each edge ⅜ deep for 26-⅜ x ¾ Hex. Hd. Cap Sc. ⅜-16-NC-3
BOTTOM PLATE
(Make from ½ x 8 cold finished flat)
Stamp Mark (each end)
EXTERIOR PARTS OF MOLD FOR BEAMS CAST ERECT (Specimens A,B,C)
Drill one ⅞ hole for Brace
Drill-2 holes
See full-size detail of this hole (upper right corner)
Note:
The size, type and location of holes at gage lines marked H, or edges marked K, are the same as detailed for the corresponding parts of mold for beam cast erect
Stamp Mark (each end)
EXTERIOR PARTS OF MOLD FOR BEAMS CAST INVERTED (Specimens AA,BB,CC)

Detail of Recessed Hole in End Plate

Values of Dimensions X Y Z for Beam Molds

Specimen	Casting Position	Length of Embedment	X	Y	Z
A	Erect	8	26½	22½	25
AA	Inverted	8	-	-	-
B	Erect	12	22½	18½	21
BB	Inverted	12	-	-	-
C	Erect	16	18½	14½	17
CC	Inverted	16	-	-	-

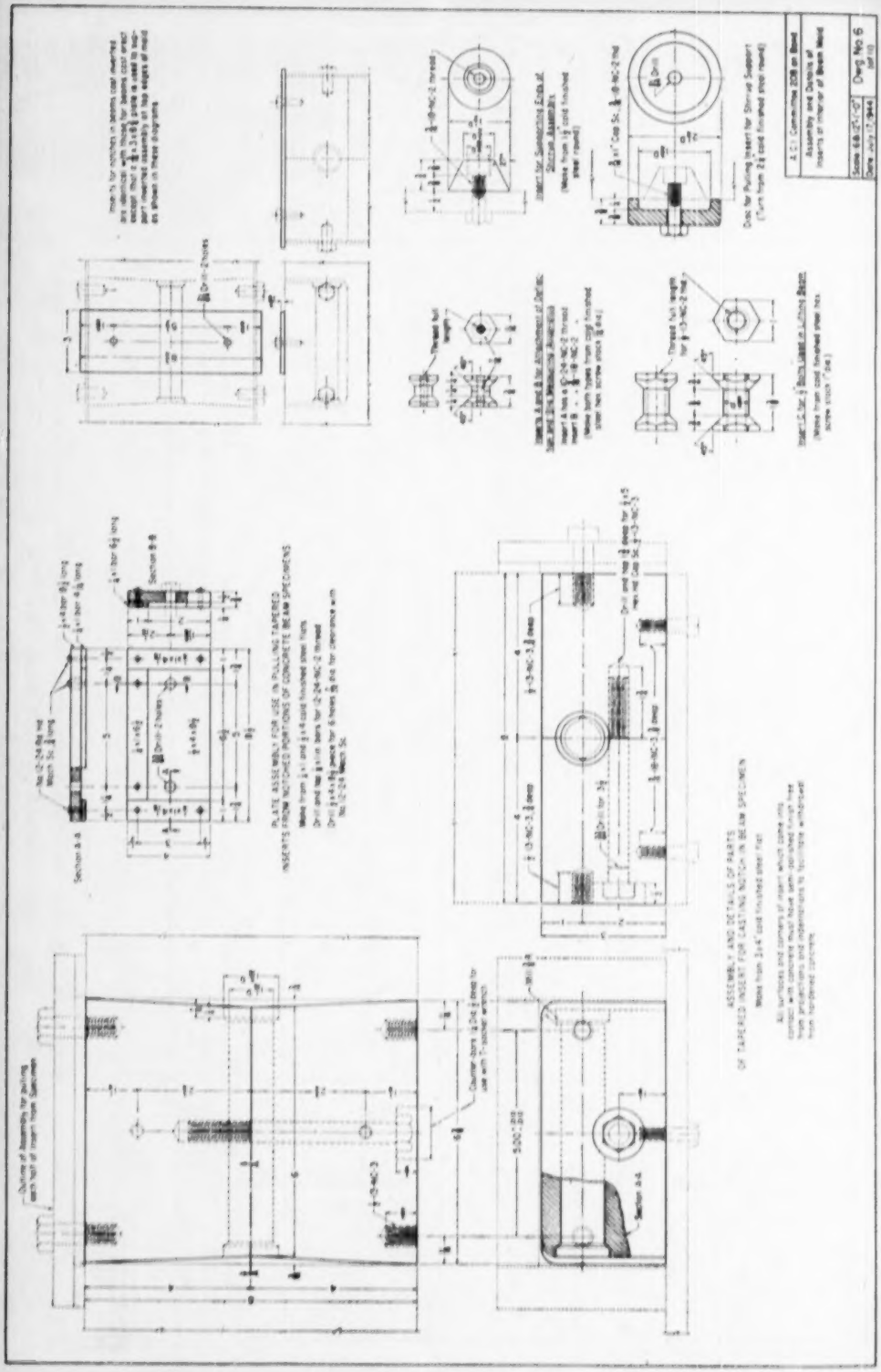
Marking System for Identifying Parts of Mold for a Given Beam Specimen

The above diagram illustrates marking of mold parts for Type C specimens. The first letter indicates the type of beam and the second letter the location of part in diagram of mold assembly. These marks are stamped into outer face of each part with steel letters ½" high. The 6 types of specimens (one complete mold for each type) are indicated in the table of dimensions above.

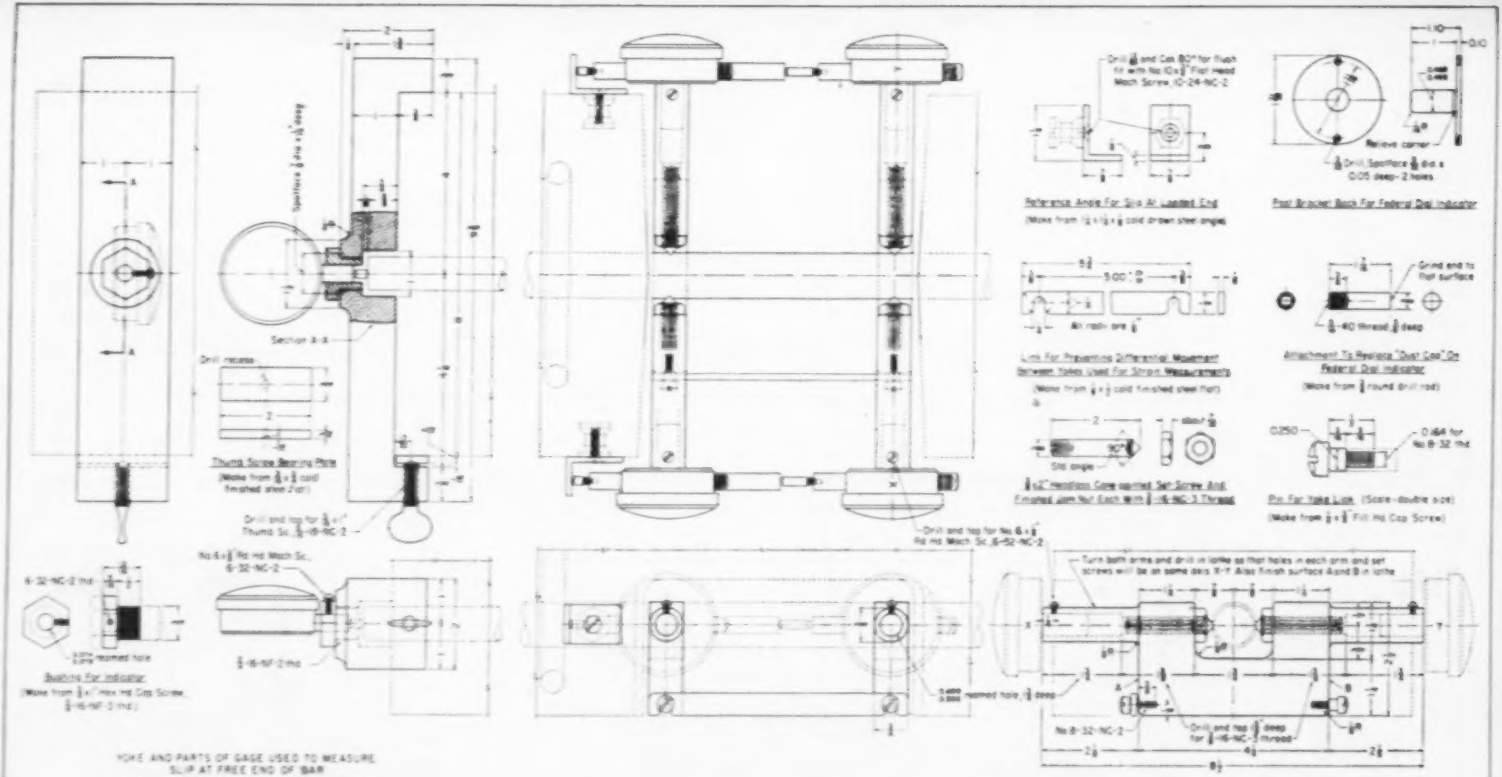
Cast-in Marks on Beam Specimens to Assist in Mounting Apparatus For Test

The pair of short dash lines detailed within circled areas of the side plates represent the location and length of V-shaped grooves milled into the inner face of each front and back plate. These grooves leave small projecting lines on the surface of the concrete and mark the correct position of bearing plates of load and reaction points during test of spec. Grooves are also milled into the inner face of all end plates to mark position of yoke of gage for slip measurements at free end of bar.

A.C.I. Committee 208 on Bond	
Details of Exterior Parts of Mold for Beam Specimens Cast Erect and Inverted	
Scale: 3" = 1'-0"	Dwg. No. 5
Date: July 15, 1944	(of 11)



Dwg. 6—Assembly and details of inserts at interior of beam mold


Section A-A
Drill recess
Thumb Screw Bearing Plate
(Make from ¼ x ¾ cold finished steel flat)
Drill and tap for ¼"
Thumb Sc. ¼-18-NC-2
No 6 x ¾ Fil Hd Mach Sc. 6-32-NC-2
¼-16-NF-2 thd
6-32-NC-2 thd
Reamed hole
Position For Indicator
(Make from ¾ x Hex Hd Cap Screw, ¼-16-NF-2 thd.)
Drill and tap for No 6 x ¾ Fil Hd Mach Sc. 6-32-NC-2
Reamed hole ¾ deep
Drill ¼ and Ctsk Bolt for flush fit with No 10 x ¾ Fil Head Mach Screw, 10-24-NC-2
Reference Angle For Slip At Loaded End
(Make from 1½ x 1½ x ¼ cold drawn steel angle)
Relieve corner
¼ Drill, Spotface ⅝ dia x 0.05 deep - 2 holes
Post Bracket Back For Federal Dial Indicator
All radii are ¼"
Link For Preventing Differential Movement Between Yokes Used For Strain Measurements
(Make from ¾ x ¾ cold finished steel flat)
Slot angle
¼-20 Headless Cone-pointed Set Screw And Fingered Jam Nut Each With ¼-16-NC-3 Thread
Grind end to flat surface
⅜-40 thread ¼ deep
Attachment To Replace "Dial Cap" On Federal Dial Indicator
(Make from ¾ round drill rod)
0.250
0.64 for No 8-32 thd
Pin For Yoke Link (Scale - double size)
(Make from ¾ x ¾ Fill Hd Cap Screw)
Turn both arms and drill in lathe so that holes in each arm and set screws will be on same axis. Y-Y Also finish surface A and B in lathe
Reamed hole ¾ deep
No 8-32-NC-2
Drill and tap ¾ deep for ⅜-16-NC-3 thread

YOKE AND PARTS OF GAGE USED TO MEASURE SLIP AT FREE END OF BAR

(Make yoke from 2" square straight drawn aluminum bar, alloy 17 ST)

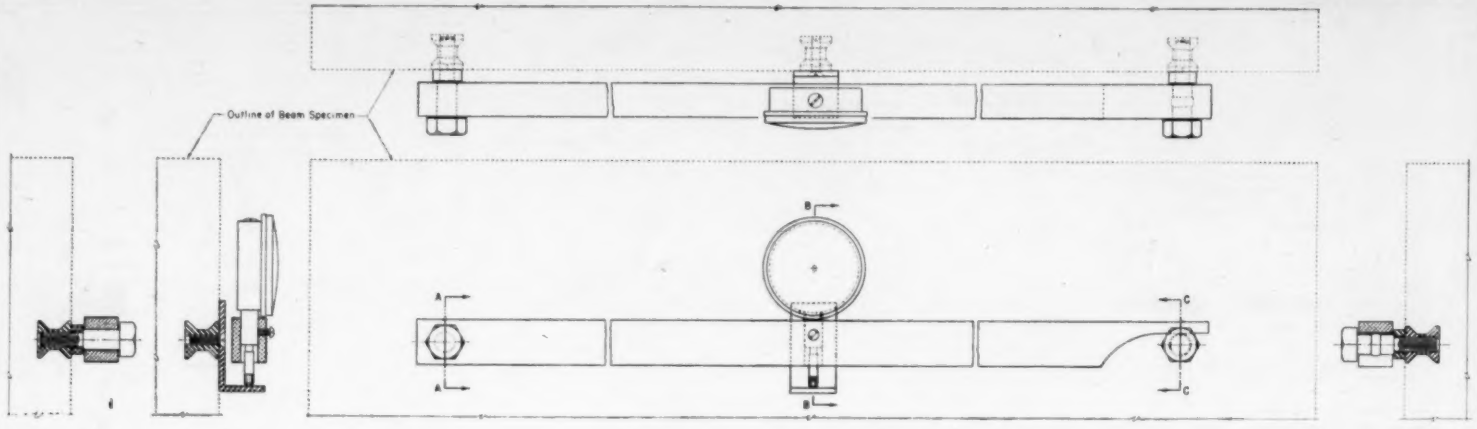
YOKES AND PARTS FOR GAGE USED TO MEASURE ELONGATION (PULL) AND SLIP AT LOADED END OF BAR

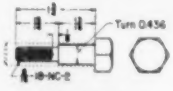
Only one link is to be used between each pair of yokes but link pins are to be provided on both shoulders of each yoke so that link can be inserted on either side of notch. The yokes in each pair are identical.

Dial indicators shown for slip and elongation measurements are Federal Model 2-D (Federal Products Corp, Providence, R.I.). They are to have continuous reading double dials with revolution counter and a range of .2000" Mech dial - a graduated for .0001" increments and a numbered 0-100 Small dial with full hole indicates revolutions. Indicators for slip at free end of bar have flat backs and ¼ long contact points. Indicators for slip at loaded end have gear bracket backs, ¼ long contact points and an attachment replacing usual "dust cap" indicators for elongation measurements have gear bracket backs and points ¼ long. Extra contact points ¼ ⅜ ½ ⅝ and ¾ long (two for each length) should be available. All points are of the regular type with ¼ dia and ¼ cylindrical

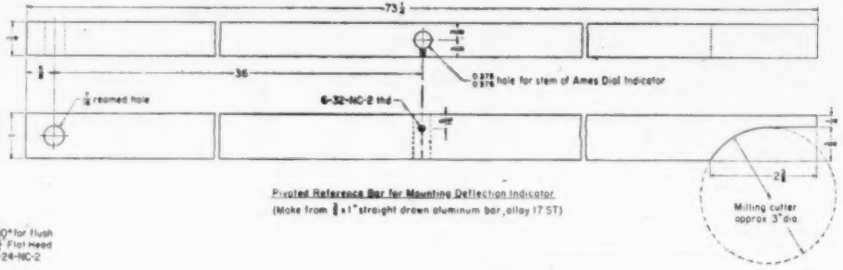
indicators should be checked and used over ranges of greatest accuracy, particularly those for elongation measurements. At least three indicators, one with flat back and two with gear backs, should be available at stations.

A.C.I. Committee 208 on Bond	
Assembly and Parts of Apparatus for Measuring Elongation and Slip of Bar.	
Scale 2"=1'-0"	Dwg No 7
Date July 25, 1940	(of 11)

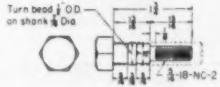




Special Bolt With Plain Shank
(Turn from ⅜ x 2" Hex Hd Cap Sc.)

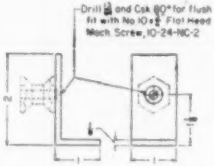


Pivoted Reference Bar for Mounting Deflection Indicator
(Make from ⅜ x 1" straight drawn aluminum bar, alloy 17 ST)



Special Bolt With Beaded Shank
(Turn from ⅜ x 2" Hex Hd Cap Sc.)

Collar For Spacing Deflection Bar
(Make from ⅜" cold finished steel round)

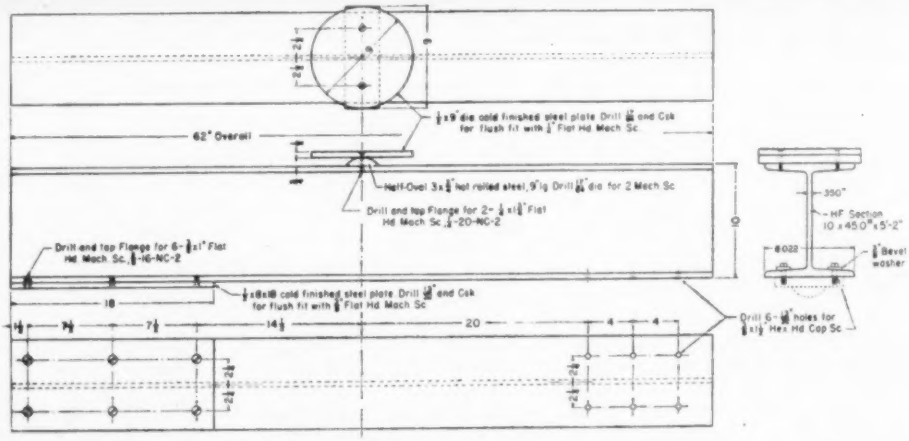


Reference Angle For Deflection Indicator
(Make from 2 x 2 x ¼ cold drawn steel angle)

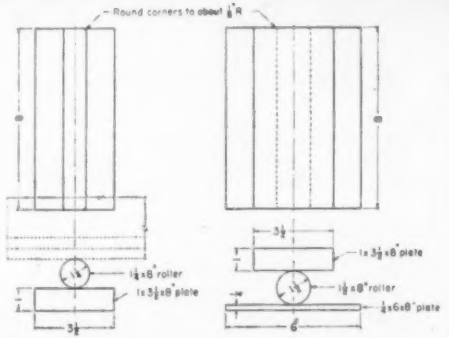
The Dial Indicator shown for deflection measurements is Ames Model 272 (B.C. Ames Co. Waltham, Mass.) provided with flat back and ½" long contact point (P-102). This model has a ½" range and continuous reading double dials with revolution counter. Main dial is graduated for .001" increments and is numbered 0-100. Small dial with telltale indicates revolutions.

A.C.I. Committee 208 on Bond	
Assembly and Parts of Apparatus for Measuring Deflection of Beam	
Scale 12"=1'-0"	Dwg No 8, (of 11)
Date July 20, 1944	

Dwg. 9—Details of apparatus for supporting and applying load to beam specimens


62" Overall
Drill and tap Flange for 6-¾x1" Flat Hd Mach Sc, ½-16-NC-2
Drill and tap Flange for 2-¾x1½ Flat Hd Mach Sc, ½-20-NC-2
Half-Oval 3x½ hot rolled steel, 9'lg Drill 11/16" dia for 2 Mach Sc
Drill 6-⅞ holes for 1½ Hex Hd Cap Sc
18
20
14½
7½

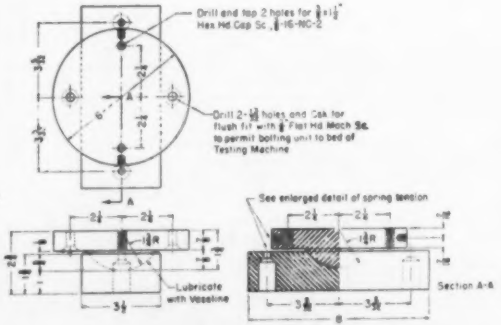
STEEL BEAM FOR DISTRIBUTING LOAD FROM HEAD OF TESTING MACHINE TO DESIRED POINTS ON CONCRETE BEAM SPECIMENS



ROLLER BEARING UNIT (J) USED AT ONE LOAD POINT

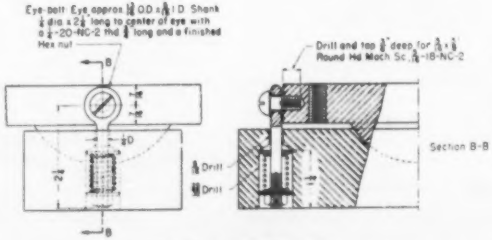
ROLLER BEARING UNIT (K) USED AT ONE SUPPORT

(Make from cold finished steel rounds and flats)


Drill and tap 2 holes for 1x1½ Hex Hd Cap Sc, ½-16-NC-2
Drill 2-1½ holes and Csk for flush fit with ¾ Flat Hd Mach Sc. to permit bolting unit to bed of Testing Machine
See enlarged detail of spring tension
Lubricate with Vaseline
Section A-A

SPHERICAL BEARING UNIT USED AT ONE SUPPORT AND ONE LOAD POINT

(Make parts from cold finished steel. Grind Spherical Bearing Surfaces to perfect fit.)


Eye-bolt Eye approx 1½ OD x ¾ I.D. Shank ¾ dia x 2½ long to center of eye with a ½-20-NC-2 thd 1 long and a finished Hex nut
Drill and tap ¾ deep for ⅞ x ⅝ Round Hd Mach Sc, ⅜-16-NC-2
Drill
Drill
Section B-B

Detail of Spring Tension Between Parts of Spherical Bearing Unit

Make spring from No 14 (.080) steel wire. Both ends to be closed and ground flat

Spring and Washer

(Scale: double size)

A.C.I. Committee 208 on Bond	
Details of Apparatus for Supporting and Applying Load to Beam Specimens	
Scale: 3/8" = 1'-0"	Dwg. No 9
Date: July 18, 1944	(of 11)

Dwg. 10—Details for machining 7/8-in. deformed bar specimens and jig for use in drilling and milling operations on bar

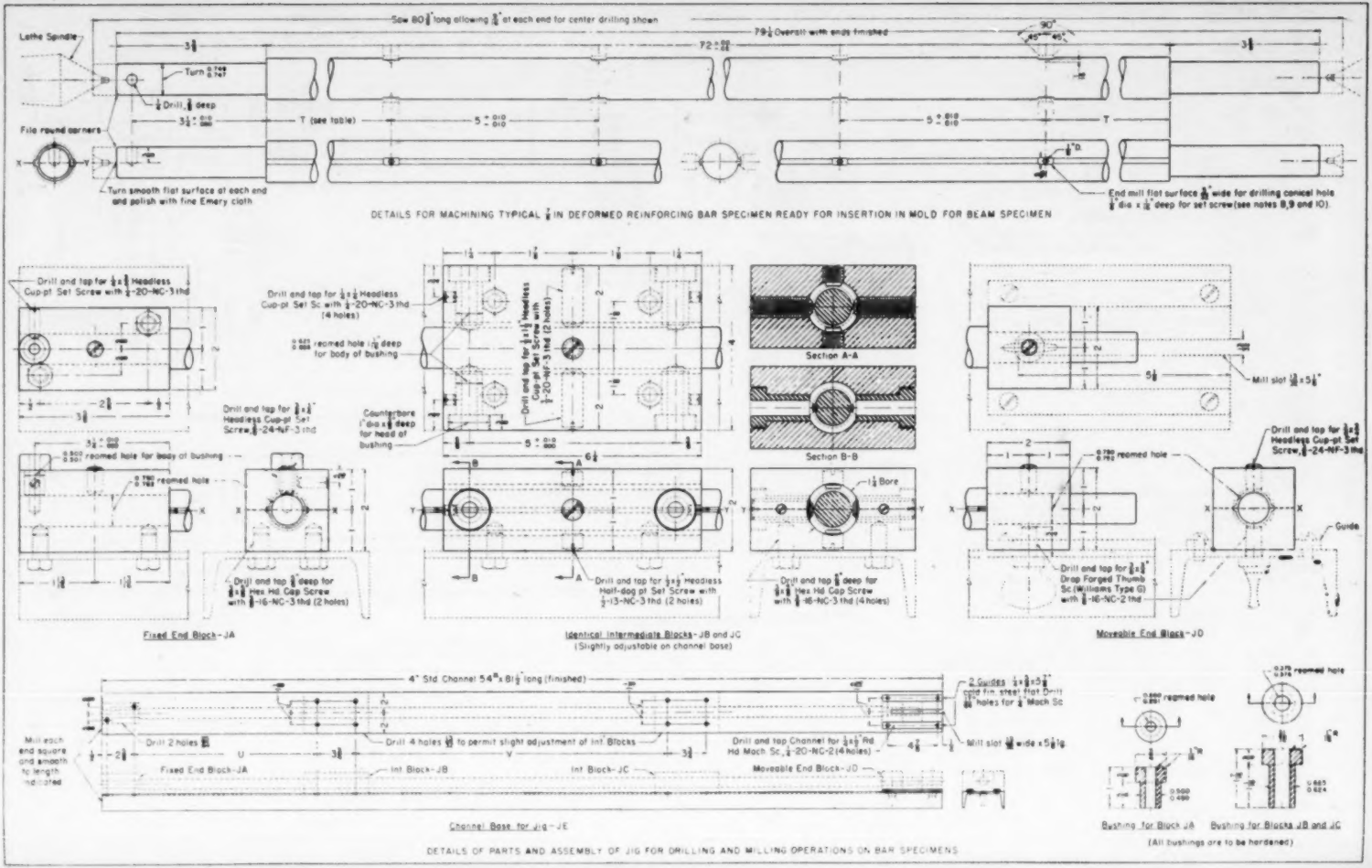

Lathe Spindle
Saw 80½" long allowing ⅛" at each end for center drilling shown
79½" Overall with ends finished
72 ± .03 .02
90°
45°
45°
3½
Turn 0.799 0.797
Drill ½ deep
3½ + .010 .000
T (see table)
5 ± .010 .010
5 ± .010 .010
T
File round corners
Turn smooth flat surface at each end and polish with fine Emery cloth
End mill flat surface ⅜" wide for drilling conical hole ⅜ dia x ⅛" deep for set screw (see notes 8,9 and 10).
DETAILS FOR MACHINING TYPICAL ⅞" IN DEFORMED REINFORCING BAR SPECIMEN READY FOR INSERTION IN MOLD FOR BEAM SPECIMEN
Drill and tap for ½ x ⅜ Headless Cup-pt Set Screw with ½-20-NC-3 thd
Drill and tap for ½ x ⅜ Headless Cup-pt Set Sc with ½-20-NC-3 thd (4 holes)
0.625 0.626 reamed hole 1⅛ deep for body of bushing
Drill and tap for ½ x ⅜ Headless Cup-pt Set Screw with ½-20-NF-3 thd (2 holes)
Countersink 1" dia x ⅜ deep for head of bushing
Section A-A
Section B-B
Mill slot ½ x 5½"
Drill and tap for ⅜ x ½ Headless Cup-pt Set Screw, ⅜-24-NF-3 thd
0.502 0.500 reamed hole for body of bushing
0.781 0.783 reamed hole
Drill and tap ⅜ deep for ⅜ x ⅜ Hex Hd Cap Screw with ⅜-16-NC-3 thd (2 holes)
Drill and tap for ½ x ⅜ Headless Half-dog pt Set Screw with ½-13-NC-3 thd (2 holes)
1½ Bore
Drill and tap ⅜ deep for ⅜ x ⅜ Hex Hd Cap Screw with ⅜-16-NC-3 thd (4 holes)
0.780 0.782 reamed hole
Guide
Drill and tap for ⅜ x ⅜ Drop Forged Thumb Sc (Williams Type G) with ⅜-16-NC-2 thd
Fixed End Block - JA
Identical Intermediate Blocks - JB and JC (Slightly adjustable on channel base)
Moveable End Block - JD
4" Std Channel 54" x 81½ long (finished)
2 Guides ½ x ⅜ x 3½ and fin sheet flat Drill ⅜ holes for ⅜ Mach Sc
Mill each end square and smooth to length indicated
Drill 2 holes ⅜
Drill 4 holes ⅜ to permit slight adjustment of Int Blocks
Drill and tap Channel for ½ x ⅜ Rd Hd Mach Sc, ½-20-NC-2 (4 holes)
Mill slot ⅜ wide x 5½ lg
Fixed End Block-JA
Int Block-JB
Int Block-JC
Moveable End Block-JD
Channel Base for Jig - JE
0.500 0.499 reamed hole
0.376 0.374 reamed hole
Bushing for Block JA
Bushing for Blocks JB and JC
(All bushings are to be hardened)
DETAILS OF PARTS AND ASSEMBLY OF JIG FOR DRILLING AND MILLING OPERATIONS ON BAR SPECIMENS

Table and notes accompanying drawing 10

Values of Dimension T for Bar and U, V for Bar Jig

Specimen	Casting Position	Length of Embed'nt	T	U	V
A	Erect	8	8.50	9.62	46.25
AA	Inverted	8	"	"	"
B	Erect	12	12.50	13.62	38.25
BB	Inverted	12	"	"	"
C	Erect	16	16.50	17.62	30.25
CC	Inverted	16	"	"	"

Reference Marks on Jig Blocks

Arrows designated "X" and "Y" point to ends of blocks in jig which are to be marked by V-grooves, shown as dash lines, to indicate center line of holes for reference in mounting bar in jig as described in notes 5 and 6 below. V-grooves should be about .02 in. wide and milled with same cutter as for V-groove detailed in DRAWING 5.

Notes on Machining of Bar

Proper machining of the Deformed Bar Specimens under investigation constitutes one of the most important steps in carrying out the test program outlined by the Committee. The following notes indicate the sequence of machining operations and the use of the jig for drilling and milling the bar. These operations should not be conducted as a "production" job but as a "precision" job.

(1) Center drill for lathe spindle each end of deformed bar specimen as nearly as possible on its longitudinal axis. If necessary, grind off lugs and seams for short distance at each end to leave round core which can be centered in chuck for center drilling.

(2) Turn ends of bar to nominal ¾-in. diameter on lathe centers. It is very important to obtain *square shoulders* at *exact spacing* for proper location in jig for drilling and milling operations.

(3) Turn ends of bar flat and to length indicated. Round corners with file and polish ends with fine Emery cloth.

(4) Install bar in jig by passing one end through blocks JC and JB into the hole of fixed end block JA and slipping movable end block JD over opposite end of bar. Block JD is clamped firmly in position by thumb screw when the turned shoulder at each end of bar is in intimate contact with the inner face of the adjacent end block.

(5) Rotate bar in its end bearings until the parting line of the upper and lower halves of the bar (as rolled) coincides as nearly as possible with the horizontal center line XX of hole in each end block (Center line is marked by V-grooves milled into inner face of each block). Cup-point set screws at the top of end blocks are then tightened to prevent rotation of bar.

(6) Half-dog point set screws at the top and bottom of intermediate blocks JB and JC are now adjusted so that the parting line of bar coincides with center-line marks YY at ends of these blocks. This adjustment is very important. It provides for alignment of bars which are not perfectly straight and controls the accuracy of milling and drilling of holes which receive the points of set screws on yokes for elongation measurements. Cup-point set screws on opposite sides of each intermediate block hold the bar firmly within the center of the hole against lateral movements during milling and drilling.

(7) When the bar is accurately positioned and firmly held at various points of the jig the ¼-in. diameter hole ⅜-in. deep at the extreme left end of bar is drilled. This hole is very important as it engages the special set screw in end plate of mold, as detailed in DRAWING 4, to control the position of the bar in the mold.

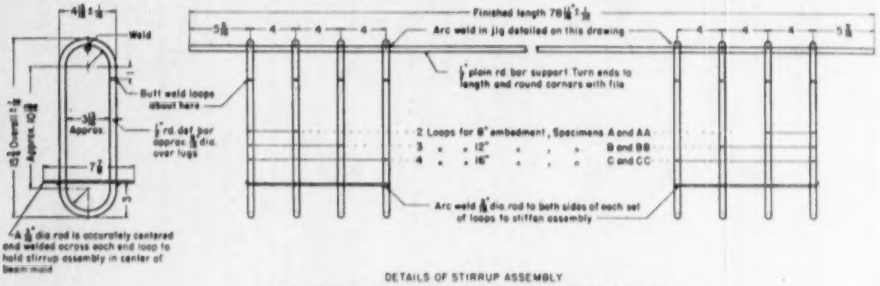
(8) Through each bushing in blocks JB and JC mill a flat surface just deep enough to provide a flat area 5/32-in. wide for drilling gage points on parting line of bar. A Putnam No. A-78 high speed end mill ⅝-in. diameter by 3¼-in. long with 4 flutes is contemplated for this operation. Keep cutting edges sharp to reduce chatter and vibration to a minimum.

(9) Through the bushings drill conical holes ⅛-in. diameter and 1/16-in. deep into flat milled surfaces of bar. These holes are to furnish firm bearing for 90° cone-pointed set screws in yokes of gages for measuring elongation and slip at loaded end of bar as shown in DRAWING 7. In drilling these holes use specially ground ⅜-in. high-speed Morse Twist Drill Company drills provided with cutting edges having a lip angle of 45° (instead of the usual 59°) and meeting in a point as sharp as possible consistent with good cutting action.

(10) All milling and drilling operations should be so conducted as to reduce the cross-sectional area of bar as little as possible consistent with achieving precision in locating firm bearings for set screws of yokes at gage points. Obviously, once a given bar specimen has been positioned in the jig it must not be moved until all drilling operations are completed. Only straight bar specimens should be selected for these tests.

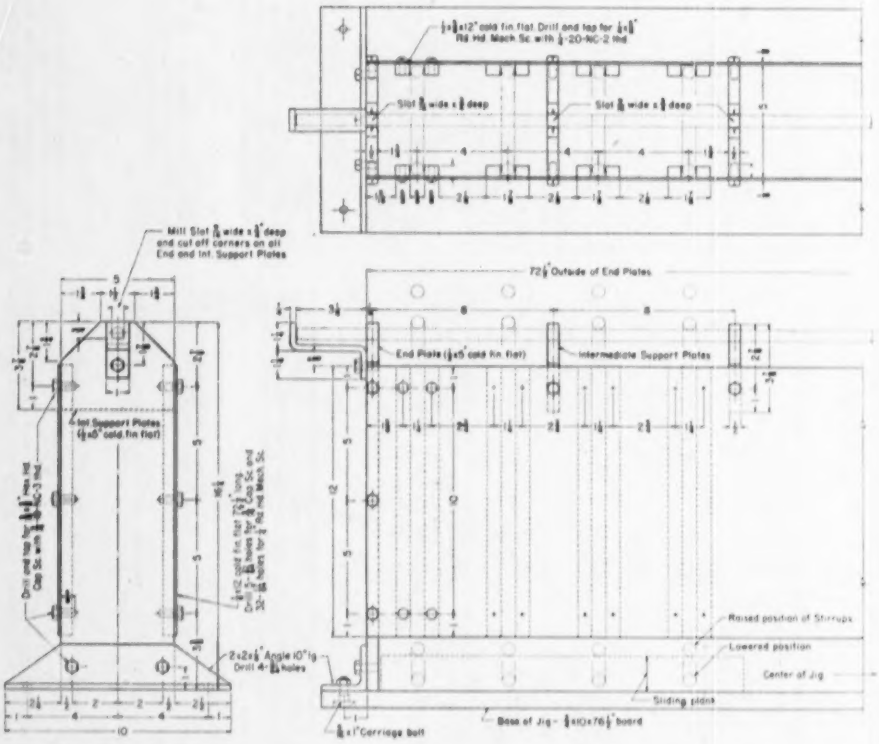
Rubber Sleeves

These are approximately 1-in. O.D., ¾-in. I.D. and 3-in. long to cover the turned portion of bar between shoulder and end plate as detailed in DRAWING 4. The internal diameter should be small enough to provide a snug fit with bar and the ends should be cut square to a length of 3-in. ±1/32.


Finished length 78⅛"±⅛
Arc weld in jig detailed on this drawing
Weld
Butt weld loops about here
Approx.
⅜" rd. def. bar approx. ⅝ dia. over lugs
Approx. 10⅛
Overall 13⅛
7⅛
A ⅜" dia rod is accurately centered and welded across each end loop to hold stirrup assembly in center of beam mold
⅜" plain rd bar support Turn ends to length and round corners with file
2 Loops for 8" embedment, Specimens A and AA
3 " " 12" " " " B and BB
4 " " 16" " " " C and CC
Arc weld ⅜" dia rod to both sides of each set of loops to stiffen assembly

DETAILS OF STIRRUP ASSEMBLY

For a given length of embedment the stirrup assembly is identical whether the beam specimens are cast erect or inverted. The number of loops at each end varies with the embedded length of the deformed bar under test as indicated above.


Slot ⅝ wide x ½ deep
Slot ⅝ wide x ½ deep
Mill Slot ⅝ wide x ½" deep and cut off corners on all End and Int. Support Plates
72½ Outside of End Plates
End Plate (½x5" cold fin flat)
Intermediate Support Plates
Int Support Plates (½x5" cold fin flat)
2x2x⅜" Angle 10" lg Drill 4-⅝ holes
Raised position of Stirrups
Lowered position
Center of Jig
Sliding plank
Base of Jig - ¾x10x76½" board
⅝x1" Carriage bolt

JIG FOR HOLDING LOOPS DURING WELDING TO HORIZONTAL SUPPORTING BAR OF STIRRUP ASSEMBLY

Use a plank about (¾x5x6)" resting on base of jig to support each set of loops until ½" bar support is in place between end brackets of jig. By sliding each plank toward center of jig the loops will drop into normal position in contact with bar ready for arc welding.

Dwg. No. 11	Bond on 302 Bars	A.S.T.M. Designation A 305-47T	Date 1948
Details of Stirrup Assembly and Jig for Holding Loops During Welding to Horizontal Supporting Bar			Scale 3/8"=1'-0"

Dwg. 11—Details of stirrup assembly and jig for holding loops during welding to horizontal supporting bar

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of the
AMERICAN CONCRETE INSTITUTE
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7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN (Supplement) November 1945

Discussion of Report of ACI Committee 208, Bond Stress:

Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars*

By DUFF A. ABRAMS and H. J. GILKEY

By DUFF A. ABRAMS†

The proposed test procedure appears to have been promulgated contrary to the experience and better judgment of Chairman Gilkey, as indicated by this quotation from the published statement:

For the pullout specimen the steel is surrounded, during test, by concrete in compression whereas in a beam the surrounding concrete is in tension. While, on the basis of limited evidence secured from tests on plain bars the chairman questions the pertinency of this difference, it has significance in the minds of many, and for such persons evidence from pullout specimens cannot be convincing.

The writer's experience agrees with Chairman Gilkey's and suggests the following 8 specific objections to the proposed tests:

1) The test procedure is very complicated and would be so expensive as to create a monopoly for the laboratory with the apparatus, skill and experience necessary to make the tests.

2) The bond value will be determined by the properties of a handful of concrete in a location where it is almost impossible to secure representative placing.

3) The bond resistance of the main bar is complicated by the high bearing stress in the concrete over the supports.

4) It is impossible properly to evaluate the effects of the 3-in. overhang at the ends of the beam.

5) The use of these types of beams to simulate typical conditions of the bond of reinforcing bars is a misdirected effort. Such beams would not be permittted by any code.

6) The notches in the tension side of the proposed beams have no counterpart in reinforced concrete beams in service; they will greatly

*ACI JOURNAL, Feb. 1945; *Proceedings* v. 41, p. 273.
†Consulting Engineer, New York City.

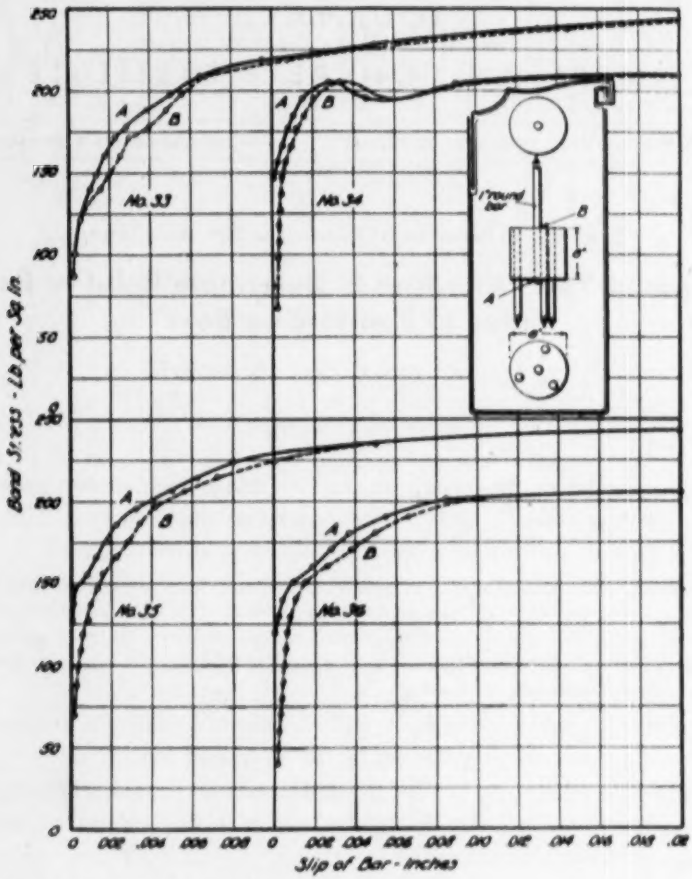

Fig. A—Load-slip curves for specimens of the form shown
(from Fig. 41, "Tests of Bond Between Concrete and Steel")

modify the normal behavior of the beams by causing vertical tension cracks to form at a comparatively low stress in the main reinforcing bar;

7) This will instantly transmit the concrete stress to the steel (as a suddenly-applied load) and cause bond failure before the required slip and stress measurements can be made.

8) The committee seems to be in much doubt as to what is needed, as shown by the 6 types of beams, which are certain to give contradictory "relative bond values" for a given bar.

The committee's statement concerning the concrete in the test beams being in tension, is, of course, correct; but this is not necessarily important and should not be decisive. However, if anyone thinks that bond tests

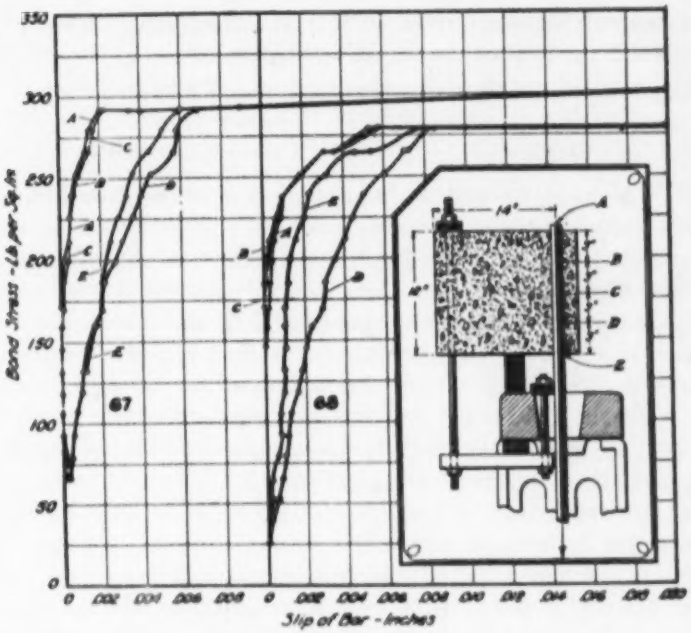

Fig. B—Load-slip curves for specimens of the form shown
(from Fig. 42, "Tests of Bond Between Concrete and Steel")

made with the concrete in tension give a better "relative bond value," a simple pull-out test can easily be designed for this purpose. This is not a speculative, untried test, nor is the idea particularly new. On the contrary, a number of different specimens of this kind were designed and used by the writer 35 years ago. Two types are shown (with representative load-slip curves) in A and B reproducing Fig. 41 and 42, from "Tests of Bond between Concrete and Steel," Bulletin 71, University of Illinois Engineering Experiment Station, 1913.

The actual stress condition near the ends of the proposed beams is much like that of the pull-out specimen in Fig. 42. In studying the load-slip curves in Fig. 41 and 42, it should be noted that this concrete had about ½ the strength of similar mixes made of modern cements, and that the plain round bars had a more rough and irregular surface than those produced in modern bar mills.

The purpose of the committee was "to determine relative bond value," but they adopted a needlessly round-about, complex and expensive test method. It is the writer's belief also that:

a) The anchorage of bars in the capitals of columns and in walls and girders gives conditions that are more nearly reproduced in the pull-out test than in the beams adopted by the committee.

b) A properly-designed pull-out test will give "relative bond values" at a cost 3 to 5 per cent of the cost of the beam tests.

c) Attempts to use the beam tests will prove disappointing.

CLOSURE By H. J. GILKEY, Chairman Com. 208

There is, perhaps, no person living more competent than Mr. Abrams to have supplied a truly searching analysis and constructive criticism by virtue of which this proposed test procedure might well have been beneficially altered; moreover the Abrams' discussion probably constitutes a fairly accurate portrayal of what the Committee should expect to be passing through the minds of many qualified critics upon a hasty first reading of the proposed test procedure. It falls short, however, of what the Committee believes the concrete public is entitled to receive from any qualified critic after he has prepared himself by studying the text and the drawings in conjunction with a careful reading of the Foreword for clarifying background and it is with distinct regret that the Committee finds itself confronted with a series of opinions, most of which are, in the light of available evidence, susceptible neither of proof nor of positive refutation. It is, with corresponding reluctance that the Committee feels forced to discuss reiteratively, questions, explanations for most of which have already been covered in the Foreword, the text and the drawings.

Mr. Abrams' opening implication of disunity within the committee is self-answered in the quotation. The purpose for which this proposed test procedure was evolved necessitates not only that we get the facts but also that those facts be convincing and accepted as facts by those whose acceptance spells use. If the technical expert for a bar manufacturer doesn't have confidence that a pullout test supplies a correct index to beam-bar bond resistance, data from pullout tests won't weigh heavily with him regardless of how thoroughly sold Mr. Abrams, the Chairman of this Committee, or any of the individual members thereof, might be on the adequacy of the pullout test as a criterion for excellence of bond performance.

Even if the item of whether the tensile bar is surrounded by compressive or tensile concrete is irrelevant (as both Mr. Abrams and the Chairman believe it to be) there are other differences between the situation existing in beams and that in pullout specimens that are not yet so easily waved aside on the basis of cleancut indisputable evidence. Settlement and water gain beneath the bar of a horizontally cast specimen, and whether the loaded end of a deformed bar projects upward or downward in a pullout specimen as cast are but two of several important differences of which, until recently, little cognizance has been taken but

which certainly cannot be thoughtlessly ignored. Abrams quotes 2 (a) (p. 275) but disregards the introductory paragraph, (b), (c) and the closing paragraphs. There was within the committee no difference of opinion with regard to pullout vs. beam-type as the primary specimen to be recommended.

Going now to the numbered items of the discussion:

1) Discussed under 3, p. 276: As pointed out on p. 273 and 274 this test procedure is not intended to be a simple acceptance test affair for use in any laboratory, possibly against a purchase specification. It is for securing basic data expected to influence manufacturing methods and processes. Compared with the importance of the information sought, the possible reduction in cost of one testing technique over another is a negligible item. Again, read No. 3, p. 276.

2) Correct in part. As in any other type of bond test, the case rests largely on the results secured from "a handful of concrete" adjacent to the bar. Good placement is one of the prerequisites of good concrete. Partially through the important contributions of Mr. Abrams himself there is no longer any justification for the harsh unbalanced mixtures of the 1910-1920 decade either on the job or in the laboratory. The committee fails to share Mr. Abrams' concern over the ability of a qualified laboratory staff to secure adequate placement for *every* "handful of concrete" in *any* portion of the beam.

3, 4) The concern over the compressive stress in the concrete around the bar appears to be a bit at variance with the introductory paragraph. Mr. Abrams may have failed to note the use of a rubber sleeve to provide clearance throughout the overhanging three inches (Drawing 4).

5) The extent to which this is or is not "a misdirected effort" is, up to the present time, a matter of personal vs. committee opinion. These are not intended to be typical beams; they are beam-type specimens designed to be stressed disproportionately high in bond in order that *bond* resistance can be measured. The Committee does not recall having seen pullout specimens "permitted by any code."

6) The extent to which notches in the concrete on the tensile face beyond the steel might induce significant localized concentration of stress is speculative. Certainly much research work has been conducted on beams cut away (usually as cast) to expose the tensile steel, accepting the established design point of view that the outside concrete functions only as "fire proofing" or "protection" for the steel. In his Bulletin 71 (University of Illinois) p. 125 Abrams employed partially cut-a-way beams which embodied discontinuities not basically different from those to which he here takes exception. He states: "Openings about 1 in. in diameter were cut or formed in the concrete under the reinforcing bar at

points where measurements of slip were desired." We know that concrete develops tensile cracks at stresses as low as 4000 psi in the steel and, as a "stress raiser," the crack itself would appear to be the pertinent item. Such cracks are present in the tensile region of any concrete beam after appreciable stress has once been applied. The Committee is inclined to brand Mr. Abrams' contention as unsupported assertion.

7) Discussion of No. 6 largely applicable. The Committee again fails to see how the short blocks of concrete (between cracks) below the bar can function either as significant loading or cushioning devices. The use of the term, "(as a suddenly-applied load)," appears to be wholly out of place and not in accord with the Structural or Mechanics-of-Materials concept and definition of "Suddenly Applied Load." The Committee reiterates that, for lack of evidence, any consideration of notch effect is necessarily speculative but, on the basis of its consideration of the notches, the Committee, unlike Mr. Abrams, does not view them with alarm.

8) To state that "The Committee seems to be much in doubt as to what is needed, as shown by the 6 types of beams, which are certain to give contradictory relative bond values, for a given bar," indicates either a lack of understanding or a disregard of the purposes of these proposed tests and the reasons back of them as outlined at some length in the Foreword. As previously discussed under No. 1 the Committee is not seeking or expecting to secure open-and-shut, yes-no answers that brand one bar as indisputably the best with the others arranged 2, 3, 4 in order of excellence. While dependable, clear-cut indications would be welcome, the Committee's sights are set only for securing basic information on the bond behavior of a few carefully selected types of deformed bars, realizing that even the best of the bars tested are likely to show some pro and con aspects. Decisions as to which bar or bars may finally be selected, and which eliminated, will involve a study of the strong points and the weak points of each of the several bars for which the data secured are, all things considered, most favorable. After the laboratory findings are available the manufacturers may be expected to weigh relative test excellence against such aspects as practicality and cost of manufacture. The number of beams deemed essential hinged upon the following considerations:

(a) *Embedded length.* If the Committee had selected a pullout test technique it is probable that it would have specified more than one length of embedment. As mentioned in the last paragraph of p. 275, the test selected does closely resemble a pullout test in certain essential respects. After careful consideration the Committee selected three lengths of embedment of the bonded portion as a desirable minimum and as the

workable maximum number of cases, the essential differences between types A, B and C being the embedded lengths of 8, 12 and 16 in. respectively as shown on Drawing 1.

(b) *Effect of settlement beneath the bar.* Probably no single aspect of bond is of greater practical importance than the effects of water gain and the settlement of the concrete from beneath any bar held in position (as bars usually are) during the plastic phase of concrete placement. This vital aspect, pointed out by Menzel only recently, failed to receive the attention it merits, at the hands of any of the early investigators (including Abrams). Surely its importance, however, cannot be minimized by Abrams or any other qualified concrete practitioner or critic. It follows, as pointed out in 7 at the foot of page 276 that no comparative investigation of bond performance of deformed bars could be considered complete if it failed to secure evidence on relative effectiveness of lugs with regard to settlement and water-gain effects. Beams AA, BB and CC of Drawing 1 (cast inverted) are introduced to cover that aspect.

(c) *Duplicate specimens.* Three duplicates to be cast for each case.

In accordance with the foregoing explanations the project calls for 18 beam-type specimens for each type of bar investigated: 3 lengths of embedment in the pullout region, 2 positions of casting—erect and inverted, and 3 duplicate specimens. Considering the basic nature of the investigation the Committee can only brand the charge of excessive numbers of specimens as being superficial. The Sub-Committee considered a much larger test program and finally proposed this as its concept of the minimum number of tests needed to give adequate information.

The Committee, and especially the Chairman, welcomes Mr. Abrams' reiteration of his 1913 observations with regard to tensile vs. compressive concrete surrounding the bar. Even here, however, Withey in Bul. 321, University of Wisconsin, "Tests on Bond Between Concrete and Steel in Reinforced Concrete Beams" (1909), makes contrary claims on the basis of tests similar to those of Abrams. From the tests by himself and colleagues, the Chairman is convinced that the Abrams finding is correct but the fact remains that even in this minor regard there is a conflict of recorded evidence, from two recognized authorities, which in itself justifies doubt in the minds of some. As pointed out earlier, this aspect was not the only, or major one, in selecting the beam-type as preferable to pullout specimens.

In his closing sentence Mr. Abrams states: "Attempts to use the beam tests will prove disappointing." Possibly so; certainly no member of the Committee expects the project to be carried out without the discovery of some "bugs." Whether disappointing or not the proposed test pro-

cedure represents the result of a lot of thorough, careful study on the part of a sub-group (with the approval of the main group) the individual members of whom have, on the basis of past performance, demonstrated considerable competence in the field of concrete design, research, and analysis. The Chairman knows of no project in the entire history of concrete research which was planned with a like degree of meticulous attention not only to the broad over-all objective but also to the minute details of execution.—On behalf and with the approval of the Committee, H. J. GILKEY, Chairman.

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Effect of Type of Bar on Width of Cracks in Reinforced Concrete Subjected to Tension*

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SYNOPSIS

Eight types of reinforcement bars were tested tc determine the effect of various kinds of deformations on width of cracks in reinforced concrete subjected to tension. The effect of repeated application of load on the width of cracks was also determined for two of the bars. The bonding efficiency of the bars was determined with a supplementary series of tensile specimens. For the most efficient type of bar, the width of cracks was less than 50 percent of that found for a plain round bar, when both bars carried a stress of 40,000 psi. A fair correlation was found between the width of cracks and the elongation at a given load measured in the tests of bonding efficiency. The width and spacing of cracks decreased, in general, with increasing bearing area of lugs on reinforcement bars.

I. INTRODUCTION

The cracking of reinforced concrete which is subjected to tensile stresses is recognized as an unavoidable evil and, in order to minimize the width of tensile cracks in some structures, designing engineers frequently use reinforcement steel in excess of the amount needed for strength. While this practice may achieve its purpose, it is possible to avoid wide cracks, with more economical use of steel, by securing better bond between concrete and steel.

The tests described in this paper were made to determine the effect of the bonding efficiency of bars upon the widths of cracks in symmetrically reinforced specimens under axial load.

The relative bonding efficiencies of the bars were determined with a supplementary series of bond tests in which the bond strength was measured by the ability of the bar to transmit its strain to the surrounding concrete.

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†National Bureau of Standards, Washington, D. C.

TABLE 1—PHYSICAL PROPERTIES OF THE REINFORCEMENT BARS

Designation of reinforcement bar	Nominal size	Diameter as determined on basis of weight per unit of length	Height of lugs	Bearing area of lugs per lineal inch of bar	Yield strength	Modulus of elasticity
		in.	in.	in.²/in.	psi.	psi.
A	⅞	0.86	0.056	0.38	47,000	28,100,000
B	⅞	.87	.068	.36	41,000	28,700,000
C	⅞	.88	.051	.22	51,000	28,200,000
D	No. 8	.91	.030	.14	76,000†	25,400,000
E	⅞	.87	.070	.20	55,000	28,300,000
F	⅞	.86	.062	.16	44,000	28,600,000
G	⅞	.87	.068	.15	57,000	28,600,000
H*	⅞	.87	—	—	39,000	29,000,000
H_1*	⅞	.87	—	—	30,000	——

*While bar *H* had considerably higher yield strength than bar H_1, both bars were of plain, hot-rolled variety and had a similar appearance. Bars H_1 were procured for supplementary tests of 5- by 48-in. cylinders subjected to repeatedly applied load, since the supply of bars *H* was found to be insufficient to complete the test program.

†Yield strength determined by "offset" method; for the other bars the "drop of the beam" method was used.

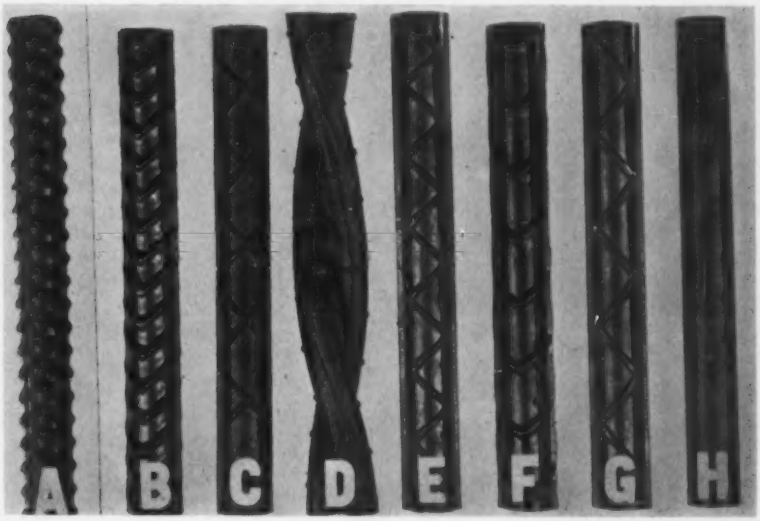


Fig. 1—Reinforcement bars

Bars of two types were also tested under repeatedly applied load and variations of width of cracks were noted under increasing and under decreasing loads.

II. MATERIALS

1. Reinforcement bars

The reinforcement bars are illustrated in Fig. 1 and their physical properties are given in Table 1. The yield strengths of the bars were determined either by the "drop of the beam" or the "offset" method, as described in A.S.T.M. Standards, Designation E 6-36, with the "offset" taken as 0.2 percent. In computing the yield point and the modulus of elasticity of a bar, its cross-sectional area was taken as the area of a circular bar of the same weight per unit of length.

Bars C, E and G represented reinforcement steel of familiar patterns which were altered either by increasing the height of lugs or by reducing the spacing between them, or by both means, while bars A, B, D and F represented bars of new design.

2. Concrete

The concrete was proportioned by weight in the ratio of 1 part cement, to 2.5 parts of sand to 3.7 parts of gravel (5.3 bags of cement per cu. yd.); the water-cement ratio was 0.65, equivalent to 7.3 gal. of water per bag of cement. The slump of the concrete was about 3 in. The aggregates were Potomac River sand and gravel; the gravel ranged in size from No. 4 to $\frac{3}{4}$ in.

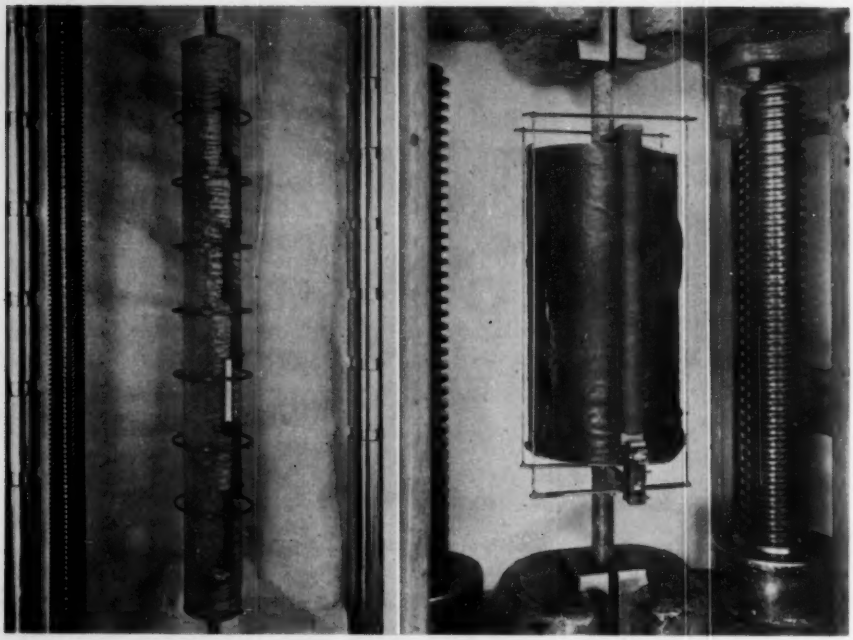


Fig. 2 (left)—Reinforced concrete cylinder in tension
Fig. 3 (right)—Bond test specimen, 6 x 12-in. cylinder, in tension

The average compressive strength for 40 cylinders of the moist cured concrete at 28 days was 3,950 psi.

III. SPECIMENS AND TESTING PROCEDURE

1. Description of specimens and of gages

The specimens for measuring the width and spacing of cracks were concrete cylinders, 5 in. in diameter and 48 in. long, with centrally placed bars. There were five similar cylinders with each type of bar. The percentage of reinforcement ranged from 2.9 to 3.1 percent for all bars except the somewhat larger bar D for which the percentage was 3.3.

The widths of tensile cracks in the cylinders were measured with fourteen 6-in. Tuckerman gages arranged in pairs along diametrically opposite elements of the cylinder (Fig. 2). The elongation indicated by a pair of opposite gages bridging a given crack was taken as a measure of the width of that crack. There were seven gages on each side of the cylinder and since the gages overlapped ½ in., the total gaged length was 39 in. The gages rested on brass strips cemented to the surface of the concrete; the brass strips had a width of ⅛ in. in the direction of the axis of the cylinder.

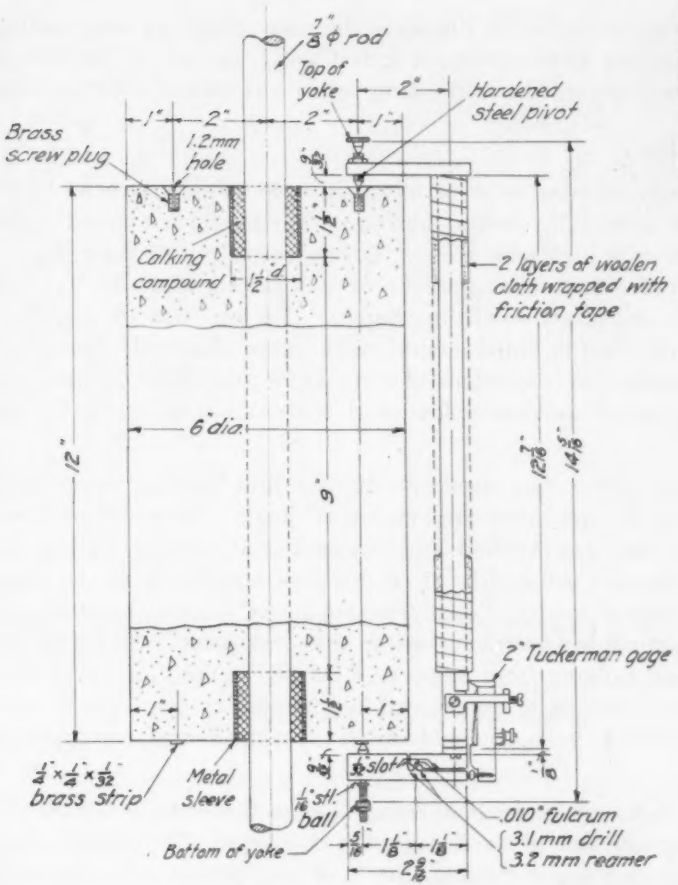


Fig. 4—Method of mounting strain transfer and gage on bond test specimen—similar apparatus installed on the opposite side of specimen

The bar A, which had the greatest bearing area of lugs per lineal inch, and the plain bar H_1 were also tested under repeated application of load using a supplementary series of five specimens for each type of bar. The specimens and arrangement of gages for these tests were identical with those described above.

The type of specimen used to determine the bonding efficiencies of the bars and one of the pair of strain gages for measuring the elongation in the surrounding concrete are illustrated in Fig. 3 and 4. There were three specimens representing each type of bar.

The strain transfers (two per specimen) shown in Fig. 4 were supported at the top by adjustable conical pointed pivots resting on brass plugs embedded in the concrete. The lower contact pivots of the transfers bore on thin brass strips cemented to the concrete. Each transfer was held in position with a flexible yoke (Fig.3) which made contact with the

strain transfer at two points in line with the pivot points of the transfer. The strain transfers were calibrated individually against a Tuckerman gage and the variation of the calibration factor was within ±0.5 percent.

2. Testing procedure

All 5- by 48-in. specimens were tested in a 60,000-lb. capacity, fluid support, Bourdon-tube, hydraulic machine. The load was applied to the ends of the bar which extended 18 in. beyond each end of the cylinder. An initial load of 100 lb. was applied and maintained while the Tuckerman gages were mounted on the specimen. The load was then applied in increments of 2,000 lb. until several cracks were observed; thereafter, the load was applied in increments of from 3,000 to 4,000 lb., depending upon the anticipated maximum load and the number of cracks in the specimen.

A similar procedure was used during the first loading cycle with specimens made for repeated application of load. For most of these specimens the load was applied and released continuously during the succeeding cycles and the widths of cracks were noted only at the maximum and minimum loads. For a few specimens the widths of cracks were also measured for both increasing and decreasing loads with increments of load ranging from 3,000 to 4,000 lb. These measurements were made during the first, second and last loading cycles. There were from 18 to 20 loading cycles applied to all of these specimens over a period of about 7 hr.

The 6- by 12-in. tensile bond specimens were tested in a 100,000-lb. capacity testing machine of the beam and poise type. The strain transfers and Tuckerman gages were mounted after applying an initial load of about 200 lb. and thereafter the load was applied in increments of about 4,000 lb.

On account of the relatively small elongations observed in concrete of the bond test specimens (maximum of about 50 x 10^{-6} in. per in.) certain precautions in guarding against thermal movement of the gages and shrinkage of the concrete were necessary. The specimens were moist cured at 70 F. for 27 days and were heated in an oven for 1 day at 80-85 F. to bring them up more nearly to the prevailing room temperature. During the heating period the specimens were protected from excessive drying with several layers of moist burlap. A rubber sleeve was slipped over the moist test specimen before it was mounted in the testing machine to minimize its drying out during the test. A waiting period of about an hour was required before a relatively stable strain gage reading could be obtained under a constant initial load. The test of the specimen thereafter was completed in from 15 to 20 minutes.

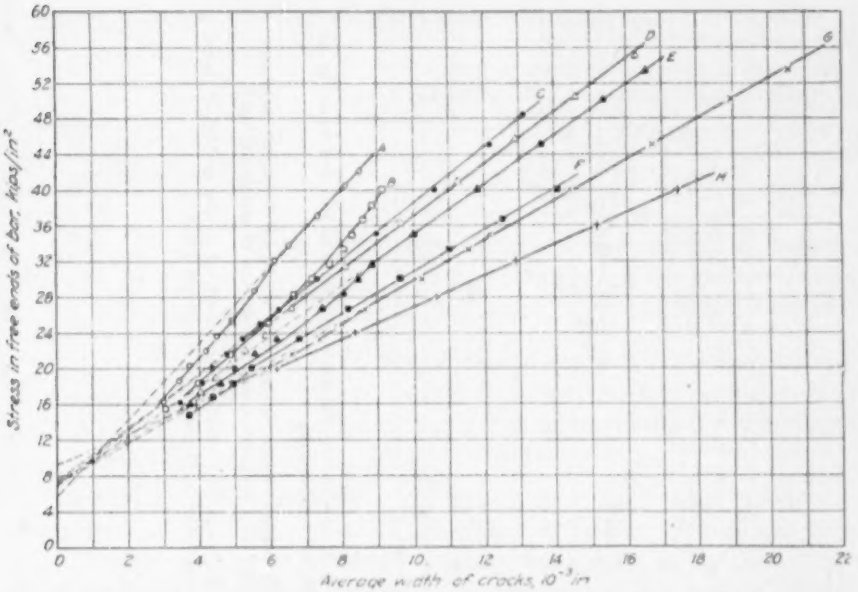


Fig. 5—Widths of cracks observed in 5- by 48-in. reinforced cylinders. (Data for the plain bar H previously published in Journal of Research of the National Bureau of Standards, V. 31, July 1943, RP1545).

IV. RESULTS AND DISCUSSION

1. Widths of cracks

The relation between the average widths of cracks and the stresses in the bars is shown in Fig. 5. For all bars except B, this relation is seen to be a linear one beyond a certain stress at which cracking of the concrete ceased and the spacing of the cracks became constant; the nonlinear behavior of the bar B may be explained by the fact that cracking continued up to the yield point with this bar.

The graphs in Fig. 5 show that bar A with the maximum bearing area of lugs developed cracks of minimum width while the plain bar H developed cracks of maximum width. At a stress of 40,000 psi., the width of cracks with the bar A was 46 percent of that with the bar H. The relative values of widths of cracks at a stress of 40,000 psi. and the stresses in various bars corresponding to the width of cracks observed in specimens reinforced with the plain bar H carrying a stress of 24,000 psi. are given in Table 2.

Comparison of Tables 1 and 2 showed that the width of cracks at a given stress generally decreased with increasing bearing area per lineal inch of bar.

Fig. 6 shows that an approximately linear relation existed between the width and the spacing of cracks at a stress of 40,000 psi. The values

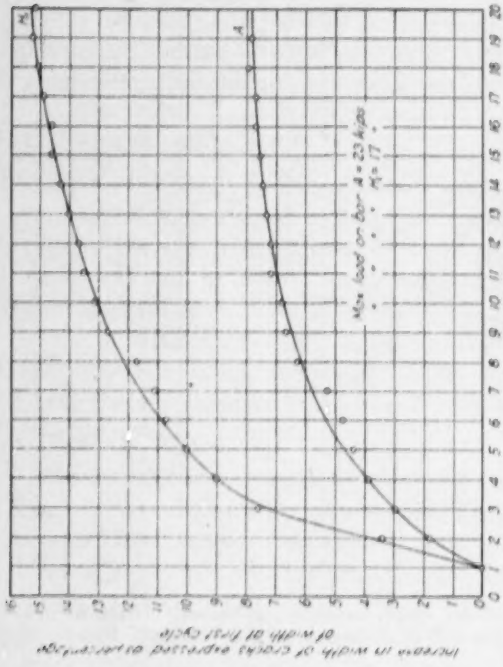


Fig. 6 (upper left)—Relation between width and spacing of cracks for various bars

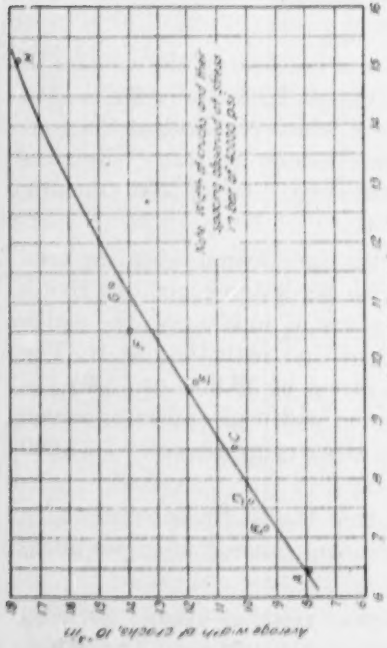


Fig. 7 (upper right)—Increase in width of cracks at maximum applied load with repeated application of maximum load.

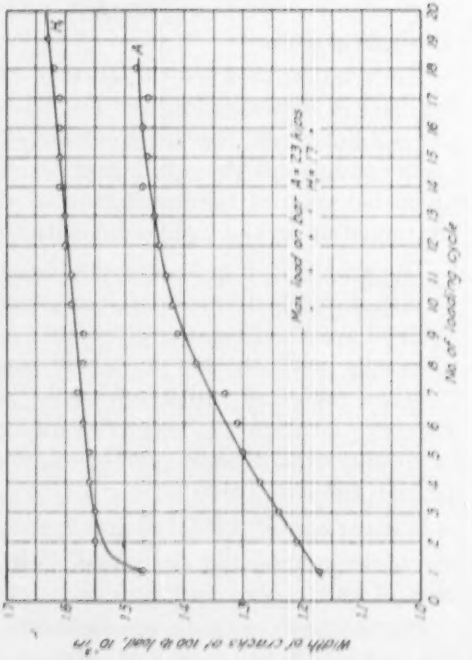


Fig. 8 (lower left)—Increase in width of cracks at 100 lb. load with repeated application of maximum load.

TABLE 2—RELATIVE WIDTHS OF CRACKS IN 5- BY 48-IN. CYLINDERS

Type of bar	Ratio of width of cracks to that observed for plain bar *H* at a stress of 40,000 psi	Stress (psi.) in bar for the width of cracks observed in specimens with plain bar *H* carrying a stress of 24,000 psi
A	0.46	41,800
B	.52	35,800
C	.60	33,400
D	.63	32,400
E	.68	30,000
F	.80	27,200
G	.83	26,200
H	1.00	24,000

of the widths of cracks in the graph were corrected for differences in moduli of elasticity, on the assumption that width of cracks is inversely proportional to the modulus of elasticity of the steel bar.

The results obtained with tensile specimens subjected to repeated application of load are shown in Fig. 7 and 8. After 19 loading cycles, the increases of the average widths of cracks over the initial values observed at the maximum loads of the first loading cycle were 7.9 percent and 15.3 percent for bars A and H_1, respectively.

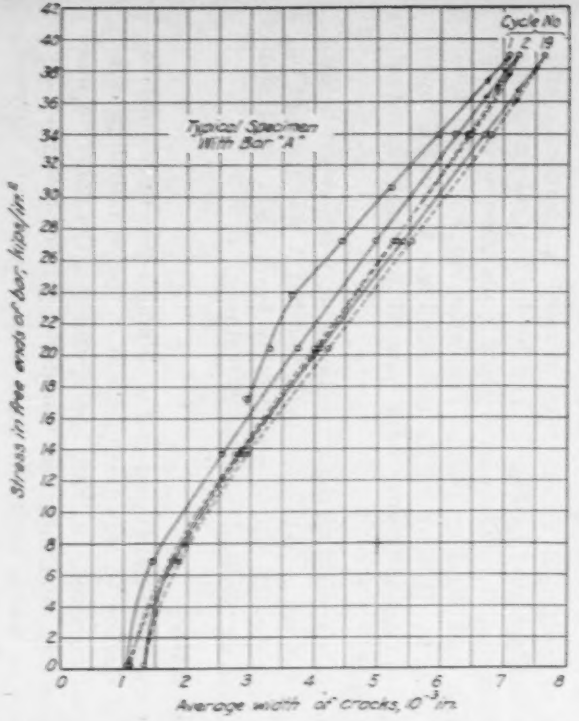
The average widths of cracks at 100-lb. load (a load of 100-lb. was maintained while minimum widths of cracks were observed) increased approximately from 0.0012 in. to 0.0015 in. for bar A and from 0.0015 in. to 0.0016 in. for bar H_1 during 19 loading cycles.

The variation of widths of cracks with alternately increasing and decreasing loads is illustrated in Fig. 9a and 9b for typical specimens representing bars A and H_1. The curves show a progressive widening of the cracks with each loading cycle. The widening was more pronounced for the plain bar H_1 than for the deformed bar A. As the repetition of loading continued, however, the rate of widening decreased for bars of both types.

The maximum loads applied to these specimens were somewhat below the yield points of the bars. While the deformed bar A was stressed to 39,000 psi., the plain bar H_1 had a considerably lower yield point and accordingly its stress was limited to about 29,000 psi.

2. Bonding efficiency of bars

The bond test specimen illustrated in Fig. 3 and 4 was developed to test the ability of the bar to transfer its strain to the surrounding concrete through bond stresses. In these test specimens, the concrete was stressed only indirectly, and thus this test differed from the generally used pull-out test in which both the steel and concrete are stressed directly by the testing machine.



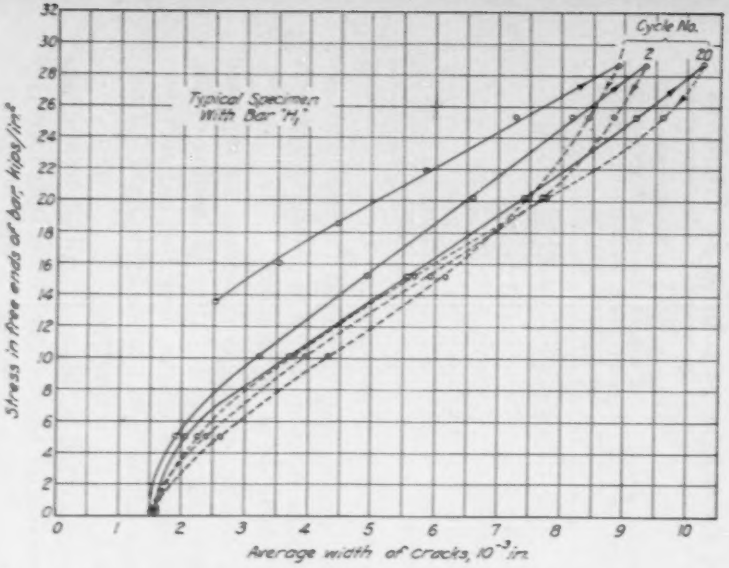


Fig. 9a (above) & 9b (top right)—Variation of width of cracks with increasing and decreasing loads.

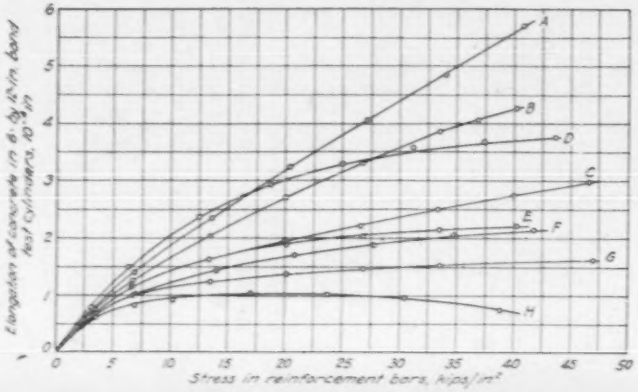


Fig. 10 (bottom right)—Elongation of concrete in bond test specimens

It will be noted from Fig. 4 that, wheras the tensile bond specimen is 12 in. long, the embedded test bar is bonded to the concrete along a length of only 9 in. The value of 9 in. was determined by trial in an effort to find the maximum bonded length of bar which would permit stressing the steel to its yield strength without causing the concrete to fail in tension. The "dummy" ends of the concrete cylinder, each 1½ in. long, were provided in order to minimize distortion of the ends of the cylinder under load. Considerable distortion or "dishing out" of the ends of the specimens was observed in similar tensile bond specimens in which the reinforcement bars were bonded to the concrete along the entire length of embedment.

The data obtained with bond test specimens are illustrated in Fig. 10. The elongation of concrete cylinders 12 in. long plotted against the stress in the bars is seen to be greatest for the deformed bar A, which had the greatest bearing area of lugs per lineal inch, for stresses exceeding 17,500 psi. It is also noted that the least elongation was observed for the plain bar H.

3. Correlation between data on cracking of concrete and bonding efficiency of bars

Fig. 11 and 12 illustrate the relation between the elongation of concrete observed in the 6-x 12-in. bond test specimens, and the spacing and the width of cracks in the concrete of the 5 x 48-in. cylinders. The curves in both graphs disclose the same general relationship between the variables, as might be expected in view of the approximately linear relation between the spacing and widths of cracks shown in Fig. 6. An increase in elongation measured on the bond test specimens was generally associated with a decrease in the width and spacing of cracks in the corresponding 5 x 48-in. cylinders.

The correlation between elongation of the bond test specimens and the spacing of the tensile cracks shown in Fig. 11 was somewhat better than the correlation of the elongation of bond test specimens with widths of cracks shown in Fig. 12, although both graphs exhibited the same general relationship.

V. SUMMARY OF RESULTS

1. Width of cracks in 5- by 48-in. reinforced concrete cylinders was least for the deformed bar A which had the maximum bearing area of lugs per lineal inch, and greatest for the plain bar H, while for the remaining bars the widths of cracks varied between these extreme values roughly in the sequence of decreasing bearing areas of lugs.

2. The widths of cracks observed for the various bars varied approximately linearly with the spacing of the cracks.

3. Widths of cracks at the maximum applied loads increased 7.9 percent and 15.3 percent for the deformed bar A and the plain bar H_1,

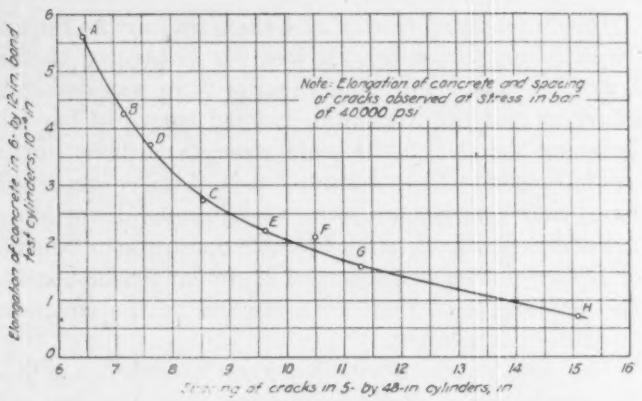

Note: Elongation of concrete and spacing of cracks observed at stress in bar of 40000 psi
A
B
D
C
E
F
G
H
Elongation of concrete in 6- by 12-in bond test cylinders, 10⁻³ in
Spacing of cracks in 5- by 48-in cylinders, in

Fig. 11—Correlation between elongation of concrete in bond test specimens and spacing of tensile cracks.

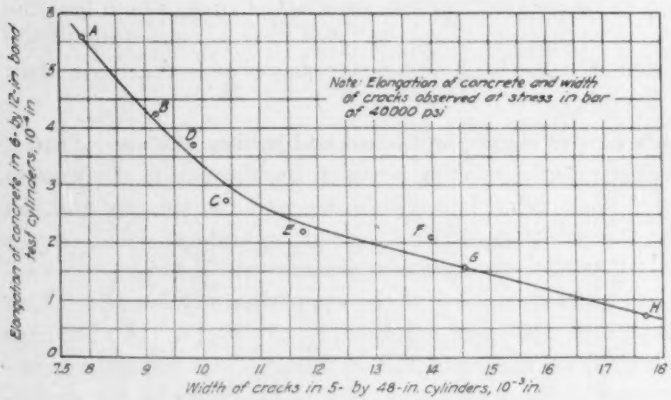

Note: Elongation of concrete and width of cracks observed at stress in bar of 40000 psi
A
B
D
C
E
F
G
H
Elongation of concrete in 6- by 12-in bond test cylinders, 10⁻³ in
Width of cracks in 5- by 48-in. cylinders, 10⁻³ in.

Fig. 12—Correlation between elongation of concrete in bond test specimens and width of tensile cracks corrected for differences in moduli of elasticity in reinforcement bars.

respectively, after 19 repeated applications of the maximum loads. The widths of cracks at 100-lb. load remaining on the bar after each application of the maximum load increased approximately from 0.0012 in. to 0.0015 in. for bar A and from 0.0015 in. to 0.0016 in. for bar H_1 during 19 loading cycles.

4. The greater the bonding efficiency of the reinforcement bars as measured by elongation of concrete in 6- by 12-in. bond test specimens, the smaller was the width of cracks in symmetrically reinforced 5- by 48-in cylinders.

Discussion of this paper should reach the ACI Secretary, in triplicate, by April 1, 1945, for publication in the JOURNAL for June 1945.

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Discussion of a paper by Watstein and Seese:

Effect of Type of Bar on Width of Cracks in Reinforced Concrete Subjected to Tension*

By J. MERCADANTE, PAUL W. ABELES and AUTHOR

By J. MERCADANTE†

The text and data in the paper admirably describe the phenomenon of cracking in concrete under tension and clearly demonstrate the advantage of high bond strength in minimizing width of cracks.

An indication of the relative efficiencies of the various types of bars considered in the paper may be obtained by the accompanying Fig. A in which the unit stresses observed in specimens to produce a given width of crack are plotted against the bearing areas of the lugs per linear inch of bar. The unit stresses are taken from the third column of Table 2 of the paper and the bearing areas from the fifth column of Table 1.

If the lines are drawn radially from the point representing the H-bar, (⅞-in. plain round) to the points representing the remaining bars, the inclinations of these lines from the vertical are a measure of the efficiencies of the patterns of the bars. This test indicates that the D-bar has the highest efficiency of the bars tested, which may be attributed to its unique twisted shape.

The possible advantages of high bond strength are now receiving intense consideration, as evidenced by new types of bar, questionnaires and investigating committees.‡

Unfortunately, at the time the tests described in the paper were made, the improved D-bar, which had been developed, was not available to the authors. The improved D-bar has transverse lugs of closer spacing and greater bearing area. Its properties, together with those of the D- and A-bars considered in the paper, are tabulated:

*ACI JOURNAL, Feb. 1945; *Proceedings* V. 41, p. 273.
†President American Insteg. Steel Corp., New York.
‡"Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars"—Report of ACI Committee 208, Bond stress.

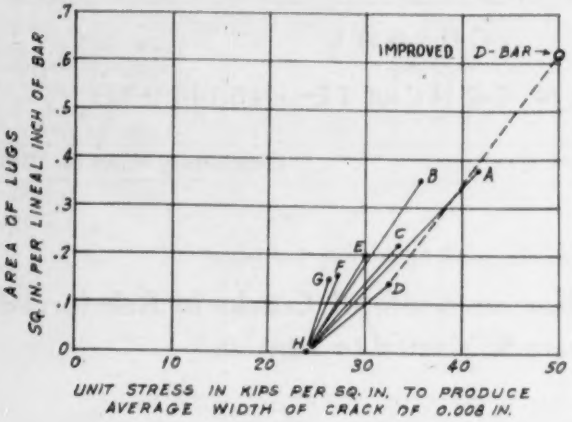

Fig. A

	⅞-in. round A-bar	No. 8 D-bar	Improved No. 8 D-bar
Cross-sectional area, sq. in.	.601	.667	.667
Bearing area of lugs, sq. in. per lineal inch of bar	.38	.14	.62
Height of lug, in.	.056	.030	.069

The improved No. 8 D-bar has not yet been tested. However, an idea of its probable performance may be obtained from the accompanying figure by projecting the point representing the D-bar of the paper along a line parallel to the average efficiency of the bars tested. This procedure indicates a possible increase in permissable unit stress over that permitted for the A-bar in the ratio of 50,000 to 42,000 without increasing the average width of crack.

It is expected that tests to be conducted in the future under the guidance of ACI committees, will also include improved D-bars.

By PAUL W. ABELES*

The authors have clearly demonstrated the gradual increase of bond of various reinforcements of old and improved pattern, dependent on the bearing area of lugs, in comparison with plain bars. From Fig. 5 and 6 it is seen that the total width of cracks per foot length, independent of width and spacing, approaches or even equals the total extension of the steel, e.g. for a stress of 40,000 psi: $\frac{40{,}000}{30 \times 10^6} \times 12 = 0.016$ in. This steel extension can be plotted into Fig. 6 by a straight line (one point of which

*Consulting Engineer, London, England.

is the experimental value of F) nearly parallel to, and close above, the graph. This means that the bond is almost totally destroyed at that stage (in F even totally).

However, at an earlier stage, the total width of cracks per unit of length is considerably reduced when deformed bars are used, since the bond is destroyed only in the immediate neighborhood of the cracks. This can be seen from the tests conducted by Dr. K. Hajnal-Konyi*). It might be interesting to mention that he was able to carry out pulling tests in which the maximum stress of the reinforcing bar in a cracked section was in excess of the ultimate strength of the steel, thus clearly demonstrating this phenomenon, which had already come to light at bending tests on beams, slightly reinforced with high strength steel. As the writer pointed out†, this phenomenon can be explained by a comparison with test results on steel bars, having notches or holes. It has been proved that in this case a stress concentration is not dangerous in a ductile material, if the reduction of the cross section extends to a very short length, reversal straining being avoided. This means that fine cracks do not cause a reduction of the concrete tensile resistance, as long as the bond is destroyed only in the immediate neighborhood of the cracks.

The writer would like to stress the authors' statement in the introduction that "cracking of reinforced concrete" . . . "is recognized as an unavoidable evil," and that wide cracks can be avoided "with more economical use of steel, by securing better bond between concrete and steel." There are three ways of increasing the bond efficiency of high strength steel:

(1) by the reduction of the diameter of the individual bar,

(2) by the provision of a special pattern at the surface (e.g. deformed bar), and

(3) by an improvement of the property of the concrete (e.g. by centrifugal molding).

The use of high strength wire is made possible by prestressing, as discussed by the writer‡. A special feature of prestressed concrete is its great resilience; that is to say, cracks occurring at a loading up to about ¾ of the ultimate load, close almost totally when the load is removed, even after repeated application of the load. A similar behavior is, within the writer's knowledge, only attained with tubular spun concrete products, having a closed transverse reinforcement which is prestressed by the centrifugal molding process.

*See Paper and Discussion on "Tests on Square Twisted Steel Bars and their Application as Reinforcement of Concrete," *The Structural Engineer*, Sept. 1943, Feb. and March 1944.

†*The Structural Engineer*, Feb. 1944, based on a lecture in Vienna in 1936.

‡"Fully and Partly Prestressed Reinforced Concrete," ACI JOURNAL, Jan. 1945.

In contrast, in conventional structures after cracking, there still remain cracks of measurable width, when the load is removed. It is interesting to note from the authors' Fig. 7 and 8 that after repeated loading the width of permanent cracks does not greatly differ whether plain or deformed bars are used, although originally at a small number of loading cycles as well as with respect to the greatest cracks, even after a great number of cycles, there is an appreciable reduction of the width in favor of the deformed bar. It can therefore be concluded that a resilience, as described above, can only be attained, if at least a part of the reinforcement is prestressed.

AUTHORS' CLOSURE

Mr. J. Mercadante's method of rating the bonding efficiencies of the deformed bars, as illustrated in his Fig. A, appears to be based on the tacit assumption that the stress in a reinforcing bar, for a given width of crack, varies linearly with the bearing area of lugs per unit of length of the bar regardless of the pattern of lugs and the shape of the bar. It would be interesting to see some experimental data which support this theory.

Mr. Mercadante's prediction of the performance of the improved bar D is apparently based on extrapolation. Extrapolation is at best somewhat speculative, and in this case leads to a conclusion which is largely of academic value.

Mr. Paul W. Abeles correctly pointed out that the total width of cracks per foot of length of the specimens approaches the extension of the reinforcing bar of the same length. However, Mr. Ables in his calculations used a value of E_s of 30×10^6 psi and arrived at the erroneous conclusion that in some cases the width of cracks per unit of length was equal to the total extension of steel of that length. When the true values of the moduli of elasticity (Table 1) were used in the calculations, it was found that in all cases the total width of cracks was substantially smaller than the corresponding extension of the steel bars.

Mr. Abeles' interesting discussion of prestressed concrete brought up the question of the effect of quality of concrete on control of cracking of reinforced concrete. In an earlier study of cracking of reinforced concrete subjected to tension.* concrete of two different strengths was used, but no significant difference between the width of cracks for a given stress and a given type of bar was observed in specimens of "weak" and "strong" concrete.

*Width and Spacing of Tensile Cracks in Axially Reinforced Concrete Cylinders, *Journal of Research* of National Bureau of Standards, V. 31, July 1943.

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Cracking and Temperature Control of Mass Concrete*

By CLARENCE RAWHOUSER†
Member American Concrete Institute

SYNOPSIS

This paper presents a discussion of certain characteristics of mass concrete which have assumed increased importance in recent years by reason of more rapid construction with modern equipment of extremely large concrete dams. Understanding of the factors affecting the temperature and the thermal stresses of mass-concrete structures is necessary if advantage is to be taken of control measures to prevent unfavorable conditions. Some of the more important factors are presented which combine to establish controlling conditions of temperature, volume change, and crack development. A section on temperature computations is included which presents methods of evaluating the effects of imposed conditions and of determining the nature and extent of the temperature control required.

FOREWORD

In 1937 a working subcommittee of ACI Committee 207, Properties of Mass Concrete, Raymond E. Davis, chairman, was assigned the task of collecting material and preparing a report covering the work of this committee. The subcommittee has collected and compiled much information on mass concrete particularly as applied to dams, and collectively and as individuals, has made field inspections which have extended from the Atlantic to the Pacific and from Canada to Mexico. The members of the subcommittee have held numerous conferences with representatives of engineering organizations interested in the design, construction, and maintenance of dams and other mass concrete structures.

While there yet remains much work to be done before the work of the subcommittee can be submitted in the form of a report for the approval of the committee as a whole, it seems appropriate that there should be released for the information of members of the Institute certain material which has been compiled.

The paper which follows, entitled "Cracking and Temperature Control of Mass Concrete," is one of such compilations. It was prepared under the direction of the subcommittee, chiefly from material furnished by the U. S. Bureau of Reclamation.—Subcommittee: R. E. Davis, F. R. McMillan, R. F. Blanks, J. W. Kelly, I. L. Tyler.

A. CRACKING

Concrete must crack whenever the tensile stress exceeds the tensile strength. In mass concrete, tensile stress is caused principally by changes in temperature. However, for a given change in temperature the resulting thermal tensile stress in different cases is not always the same, but is modified by variations in coefficient of expansion, effective or sus-

*Received by the Institute, Dec. 29, 1944.
†Engineer, Bureau of Reclamation, Denver, Colorado

tained modulus of elasticity, and degree of restraint. The numerical magnitude of thermal stress in any case is the product of the four quantities: temperature change, coefficient of expansion, effective modulus of elasticity for the particular pattern of stress or load, and degree of restraint. It is evident, then, that the cracking tendency is least when

(1) the change in temperature is small,

(2) the coefficient of thermal expansion is low,

(3) the sustained modulus of elasticity is low, as at early ages or at any age when temperature change is slow,

(4) there is a low degree of restraint, and

(5) tensile strength of the concrete is high.

The change in temperature usually associated with cracking tendency of concrete is a reduction in temperature. Therefore, measures which prevent a large reduction in temperature are usually effective in preventing cracks. A simple expedient is to place concrete at a sufficiently low temperature so that the maximum temperature attained with rise due to heat generation and absorption will not be materially above the final stable temperature, so that little or no decline in temperature from the maximum will result. This can be accomplished most readily by placement during cold weather.

Thermal expansion or contraction is influenced principally by the type of aggregate. The thermal coefficient of concrete is generally lowest for some limestones and basalts, intermediate for granites and sandstones, and highest for quartzites and cherts. The coefficient of thermal expansion determined from laboratory tests ranges from 0.000 004 to 0.000 007 per degree F.

"Sustained modulus of elasticity" is a term used to include the elastic and plastic effects in one expression to aid in visualizing the net effects of stress-strain conditions up to any given time. The sustained modulus at any time is computed by dividing the unit sustained stress by the sum of the elastic and plastic deformations at that time. Obtained in this manner, the sustained modulus represents a definite property of the concrete and as such may be used in comparing the behavior of different concretes. In general, a low value is desirable in order that differential stresses and possible cracking may be low. Concrete in service is not usually subjected to constant stress in the manner in which the sustained modulus is obtained, so that the sustained modulus may not be directly applicable for determining stresses. However, the elastic and plastic effects can be combined in an overall stress-strain relationship in the service structure which might be called the "effective modulus of elasticity." The value of the effective modulus at any time will depend upon the entire stress history. The value for any particular

time-stress pattern may be determined in the laboratory by tests which reproduce a similar stress or strain pattern.

The degree of restraint is defined as the fractional part of the potential length change that is prevented from taking place. When concrete is permitted to expand or contract freely, no stress results from a change in temperature, but when the expansion or contraction is prevented, or "restrained," to a greater or lesser degree, some stress must result. For example, if concrete rises in temperature one degree and the coefficient of expansion is five millionths per degree, the potential length change is five millionths. If then the concrete is permitted to expand only two millionths, the degree of restraint is 60 percent, because 60 percent of the desired movement is prevented.

The number of degrees of change in temperature is only one of several factors which determine the cracking tendency of mass concrete. It is almost meaningless to determine the number of degrees of temperature drop a concrete can withstand without considering these other factors. A convenient constant, however, is the amount of *sudden* temperature drop which can be withstood when the restraint is 100 percent. This constant is usually assumed to be about 20 F. because ordinary concrete is considered to have a tensile strength of not much over 300 psi. There is good indication, however, that the true tensile strength of concrete may be about double this value. Actual values, of course, depend on the class of concrete. These numerical values should be considered to be relative only, although they do represent average values for present-day concrete. The gradual temperature drop which can be withstood by concrete is about twice the "sudden" value. A gradual drop is considered to be one which occurs slowly over several weeks' time. It is not rare to find that the temperature drop which concrete is able to withstand may be as much as *three* or *four* times as great for a very slow change as for a sudden change. The reduction in sustained modulus of elasticity accounts for about two-thirds of the benefit due to slow change, and the remainder is due to the more general participation of the concrete mass and its foundation in the temperature change, thus reducing the degree of restraint.

The main sources of cracking in mass concrete are (1) from internal joints, galleries, or temporary bulkheads, (2) from the junction between concrete and foundation (rock or cooled concrete), (3) at construction surfaces when exposure conditions are severe, and (4) at the side surfaces which have been formed and which later are exposed directly to weather. The last named is probably the source of most cracks.

Internal joints, galleries, or bulkheads are obviously conducive to cracking. Not only do they reduce the section which must carry the tensile stress due to cooling, but they concentrate stresses at their edges.

The indicated precaution is to avoid galleries and bulkheads in regions where cracks are likely to occur, or at least in regions where cracks would be detrimental to structural stabilty.

Thermal stresses are severe near the junction between concrete and foundation because large temperature changes normally occur in the concrete in this region where the restraint is high. Breaks in the foundation or abrupt changes in the contact surfaces are especially objectionable points of concentration of stress. A transition zone of temperature change which reduces the stresses can be effected by casting several shallow lifts of two or three feet in depth on the foundation, allowing each to cool for several days before adding the next. The temperatures in this zone can be controlled even more effectively by the use of artificial cooling with closely spaced pipes. It should be appreciated that an artificial foundation results at any horizontal joint in the concrete where resumption of concreting is delayed for several weeks, and there conditions are almost as severe as at the rock foundation itself. For this reason, regularity of casting concrete is important.

The most difficult source of cracking to control is at formed surfaces. In addition to cooling from the maximum temperature which was caused by heat generation of the cement, the weather fluctuations are added. Here, too, the surface often undergoes drying shrinkage which causes tension at the surface and helps to start cracks. The type of forms and time of removal of forms are important factors which are discussed in a following paragraph. Further discussion of experiences in crack control is contained in reference (1).

Nevertheless, temperature change is the most significant of the factors which together influence the cracking tendency of mass concrete and, fortunately, a factor which is subject to practical and economical measures of control. In the following sections this matter is discussed in some detail.

B. FACTORS AFFECTING TEMPERATURE CONDITIONS AND CONTROL

Careful control of temperature is required if volume changes are to be kept sufficiently small so as to prevent significant cracking of mass concrete. Consideration of the temperature variations without control and computations of the effects of variables present factors of control and permit their evaluation. The problem is usually that of treatment of excess heat, and most methods of control seek either to prevent or remove such excess heat. Evaluation of the effect of the many variables permits choice of the method which is most effective for the least cost. Some of the more important factors which combine to establish controlling conditions of temperature, volume change, and crack development may be listed as follows:

(1) Climatic conditions
  (a) Concrete placing temperatures
  (b) Conditions of exposure during construction
  (c) Conditions of exposure during operation
  (d) Final stable temperature of concrete
  (e) Seasonal variations in temperature
(2) Composition of concrete
  (a) Water-cement ratio
  (b) Type and amount of cement
  (c) Type and character of aggregate
  (d) Thermal characteristics of concrete
  (e) Physical properties of concrete
(3) Requirements of design and construction
  (a) Size and shape of section
  (b) Contraction joints
  (c) Forms and form removal
  (d) Curing with water
  (e) Special treatment to reduce placing temperature
  (f) Artificial cooling
  (g) Other expedients to control temperature
(4) Rate of construction
  (a) Thickness of lifts
  (b) Time interval between lifts
  (c) Time of exposure at contraction joints
  (d) Seasonal limitations for placing concrete
(5) Character of foundation
  (a) Temperature
  (b) Thermal characteristics
  (c) Profile and preparation

Discussion follows of the modifications in the temperature conditions which may be obtained by varying the individual factors listed above.

1. Climatic conditions

a. Placing temperature—The temperature of the concrete at the time of placement depends chiefly on the temperature and specific heat of the various ingredients. Without definite control it is usually very close to the mean monthly air temperature. The temperatures of the aggregate in the stock pile and of the cement and mixing water do not ordinarily follow the daily air temperature fluctuations. Reduction of the placing temperatures by reducing the temperatures of the concrete ingredients offers a method of control by removal of excess heat before it is entrapped in the structure. Lowering the placing temperature also reduces the rate of heat generation at the early ages and thus facilitates control of the temperature rise. In a hot, dry climate the temperature of the coarse aggregate can be reduced close to the wet bulb temperatures by sprinkling and evaporative cooling just prior to use. Where the plant arrangement is such that the coarse aggregate can be sprinkled, this offers an economical way of removing a fair amount of excess heat. The benefit to be derived in lowered concrete temperature by lowering the temperature of the various ingredients is indicated in the comparison below. The mix represented is that of the mass concrete of Friant Dam. The mix proportions by weight, of water, cement, pumicite, sand, and coarse aggregate are 0.79:1:0.25:3.37:9.35.

Comparison Showing Effect of Temperature of Ingredients on Placing Temperature

Without heat loss to or gain from the outside, to change the initial temperature of the concrete of Friant Dam 1.0 degree F.,
the 9.35 parts of coarse aggregate with attendant 0.050 parts of moisture must be changed 1.9 degrees or,
the 3.37 parts of sand with attendant 0.296 parts of moisture must be changed 3.7 degrees or,
the 0.395 parts of added mixing water must be changed 8.7 degrees or,
the 1 part of cement must be changed 18.1 degrees or,
the 0.25 parts of pumicite with attendant 0.049 parts of moisture must be changed 35.7 degrees or,
with mixing water at 32 F., 6.0 percent of the mixing water, which amounts to 0.024 parts, must be replaced by ice.

From the comparison above, it follows that if all the mixing water at 32 F. is replaced by ice, the reduction in temperature of the concrete will amount to 16.7 degrees. It is also seen that very little benefit is to be derived by lowering the temperature of the pumicite or the cement. It is usually not easy nor economical to change the temperature of the sand. Lowering the temperature of the coarse aggregate or the mixing water or replacing part of the mixing water with ice is effective in lowering the temperature of the concrete. Unless special precautions are taken, considerable heat will be gained in the acts of mixing and placing if the mixture is very much lower than the air temperature. Heat will also be absorbed during the early age by the concrete if the initial temperature is much lower than the exposure temperature, so that the reduction in maximum temperature is ordinarily much less than the reduction in initial temperature. When the initial temperature is lowered and artificial cooling with embedded pipes is started immediately, the temperature of the concrete at the bottom of the lift in contact with the rock or the previously placed concrete can be controlled much more effectively than by either method used independently. When this is done it is desirable to place the lift as rapidly as possible to reduce absorption of heat from the surface.

b. Conditions of exposure during construction—During construction the top of each lift is exposed during the interval between lifts, with the result that the surface temperature is likely to be close to the exposure temperature which, with continuous water curing, may be even lower than the placing temperature. Without artificial cooling, the temperature near the bottom of the lift will rise almost adiabatically for the first two days with the result that a steep temperature gradient with depth is developed through the lift. Peak temperatures will usually be reached in a 5-foot lift in about three days after placing. Optimum conditions for reduction of temperature by exposure of the surface of the lift without development of excessive thermal stresses may be reached

with exposure of about 8 or 10 days. Further exposure will result in large temperature change of the top lift which, if restrained by the foundation or the lift below, will be conducive to development of tensile cracks in the exposed surface. In a similar manner, the sides of blocks are often exposed for long periods during construction to lowering exposure temperatures, with development of steep temperature gradients through the block which frequently results in crack development in the sides of such exposed blocks. With continued lowering interior temperatures, such cracks in the sides and across the top of exposed blocks may extend entirely through the block and, if resistance to load stresses is dependent upon monolithic action of the block, may affect the stability and safety of the structure.

c. Conditions of exposure during operation—Following construction, heat can be lost only to the surfaces or to openings such as galleries or outlets through the mass. In the case of dams, with full reservoir, the upstream face below water level remains at fairly uniform temperature throughout the year. The downstream face and that part of the upstream face which is above the water surface are subjected to the daily and seasonally fluctuating atmospheric temperatures and, depending upon the orientation, to the effects of direct sunlight. These exposure conditions are all cyclic in character, with mean temperatures which are nearly constant over a period of years. Unless the dam has been artificially cooled during the construction period, or unless the concrete has been placed at low temperatures, the temperature of the interior at the time of completion will be considerably higher than the mean exposure temperature. This excess heat will be lost gradually during operation. The period of gradually lowering temperatures will vary from only a few years for a thin dam to many years for one of thick section; the time required to lose a given percentage of excess heat under similar conditions varying as the square of the thickness. During this period, therefore, the interior of the dam may be lowered through a much larger range of temperature than the concrete near the surfaces, inducing temperature stresses across the section. If these stresses are within the strength of the concrete they may not be objectionable. The gradual lowering of the temperature of the interior of the mass may produce a more favorable distribution of total stress across the section. However, over-all contraction due to temperature reduction will cause contraction joints to open and, if the change from maximum to final stable temperature is excessive, may produce cracks in uncontrolled locations and directions. Such cracks may affect the durability and safety of the structure.

Where penstocks or outlets pass through the dam, reservoir water temperatures are carried through the section in what are, in effect, large cooling pipes which may materially affect the temperature distri-

bution through the section surrounding the pipes. Unless galleries and shafts are open to the surface in such a way as to permit free circulation of air or produce chimney effects, the presence of such openings will not materially affect the temperature distribution throughout the section.

d. Final stable temperature of concrete—Any dam must eventually assume a temperature dependent on the mean annual air and water temperatures to which the structure is exposed, as modified by direct solar radiation on the dam. It is not feasible nor scarcely even possible to control the state of temperature equilibrium which a structure will attain. A fairly close estimate of this temperature state must be anticipated if the concrete temperatures are to be controlled to conform thereto. To make such an estimate requires information of the air and water temperatures at the dam site. Information on air temperatures is usually obtainable from climatological data from some nearby Weather Bureau station. Water temperatures may be obtainable from records in connection with previous development on the stream, as from power plant operation records, or they may have to be estimated from the air temperatures and humidity records.

The effect of direct sunlight on the exposed surface may be of importance. Observations indicate that there is seldom as much heat lost from the surface as is computed when surface temperature is considered to be equal to the air temperature to which the surface is exposed. Other factors may also affect the losses slightly but it is certain that solar radiation plays an important part. Observations have been made at Boulder Dam of the effect of solar radiation on the surface temperature of the concrete. At one point on the downstream face the surface temperature is above the mean air temperature by an average of from 4 degrees in midwinter to about 16 degrees in late summer. At a point on the vertical upstream face where both the duration of sunlight and the intensity on unit area are reduced from that on the downstream face, the increase in surface temperature above the mean air temperature varies from 1 degree in midwinter to 11 degrees in midsummer. These variations in surface temperature from the air temperature must be considered in arriving at an estimate of the final stable temperature of the structure.

e. Seasonal variations in temperature.—Ordinarily the seasonal variations affect only the outside shell of a massive structure, the daily variations being negligible at depths greater than 2 to 3 feet, and the seasonal variations being unimportant at depths greater than 20 to 30 feet. In a thick dam, therefore, the cyclical daily and seasonal variations are not of importance, except as such variations in surface temperatures may contribute to surface cracking and deterioration. In a thin dam

exposed to large changes in temperature, as Seminoe Dam in Wyoming which is 15 feet thick at the top and 90 feet thick at the base of its 295-foot height, the seasonal variations may be of importance throughout the greater part of the structure. With the upstream face of the dam exposed by drawdown of the reservoir, the upper 100 feet of Seminoe Dam, throughout its entire thickness, during late winter has a temperature lower than the freezing temperature of water. This condition definitely affected the time when the contraction joints could be grouted, and cooling and grouting plans had to be made to conform to these temperature conditions.

2. Composition of concrete

a. Water-cement ratio—The water content of the concrete mix is determined largely by the requirement for workability of the fresh concrete. Strength and other requirements, such as impermeability and durability, then determine the water-cement ratio and thereby the cement content. The major effect of choice of water-cement ratio on the temperature control problem is the extent that such choice determines the cement content. A minor effect, usually not of sufficient importance to be considered, is the change in thermal characteristics caused by variation in water content.

b. Amount and type of cement—An effective factor in temperature control is the amount and type of cement used. The temperature rise of the concrete due to heat of hydration of the cement may be considered to vary directly with the cement content. Increasing the fineness of the cement increases the amount of heat developed at early ages by about the same percentage that the early strength is increased. The type of cement plays an important role in the total amount and in the rate of temperature rise due to heat of hydration. The curves of Fig. 1 show average values of adiabatic temperature rise during the first 28 days in mass concrete for various types of cement. Comparison of computed average temperature rises of 5-foot lifts of concrete containing normal cement and low-heat cement and exposed at the top surface to the initial temperature for seven days is also shown in Fig. 1. Concrete of average thermal characteristics containing one barrel of cement per cubic yard was considered in this comparison. A lower maximum temperature of the concrete is obtained with low-heat cement due to the slower rate of heat generation during the early age, since the rate of cooling will more quickly become equal to or can more readily be made to equal the rate of heat generation.

The type of cement alone, without consideration to other factors, has a very small effect in controlling the total temperature decline from maximum to final temperature in a given concrete structure of massive

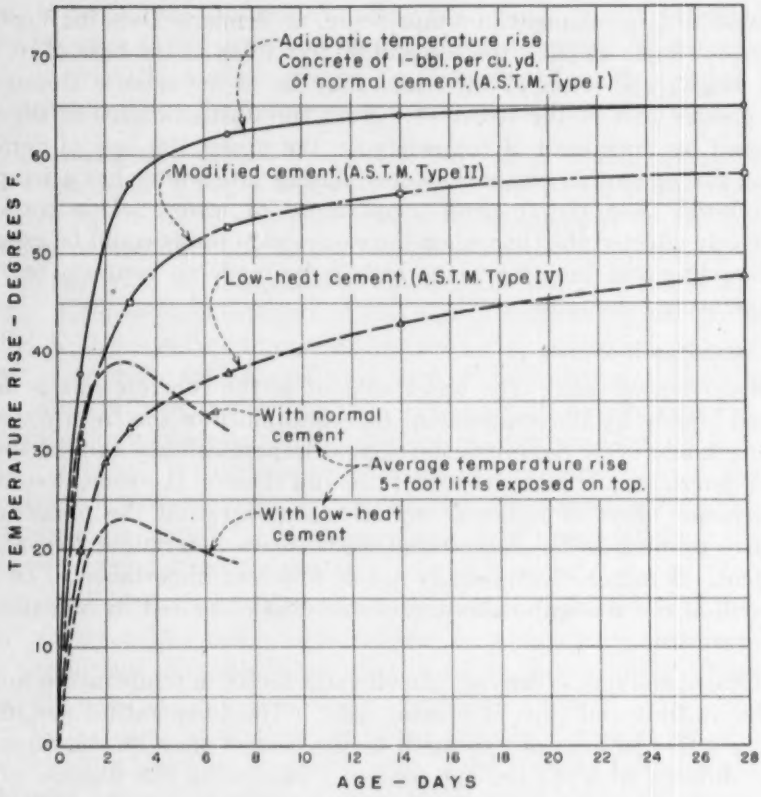


Fig. 1—Temperature rise in mass concrete for various types of cement

proportions. The total excess heat is made up of two components: (1) that amount by which the placing temperature exceeds the final temperature and (2) that due to heat of hydration of the cement. Usually the placing temperature has a much more important bearing on the temperature decline and consequent contraction of the concrete than any change that may be effected by varying the heat-generating characteristics of the cement. However, in structures of smaller dimensions, such as thin arch dams or the buttresses of multiple-arch dams wherein the rate of heat loss equals or exceeds the rate of heat generation at relatively early ages, the average maximum temperature attained by the concrete might be reduced as much as 10 to 15 degrees F. by the use of low-heat cement as compared with normal cement.

The most effective use of low-heat cement in control of temperature is in conjunction with artificial cooling with closely spaced cooling pipes. The slower rate of heat generation with low-heat cement permits the

removal of heat almost as fast as it is developed, so that temperature rise of concrete midway between pipes spaced 2½ feet apart can be limited to 5 or 6 degrees.

Puzzolanic material is sometimes added to portland cement and the mixture then used to produce concrete which has somewhat the same properties as corresponding concrete made with low-heat cement. The effectiveness of the use of portland-puzzolan cements to control the temperature depends on the percentage and type of puzzolanic material and the type of portland cement used.

c. Type of aggregate—The thermal characteristics of the aggregate, which vary with the mineral composition, determine to a large extent the thermal characteristics of the concrete. Though it is seldom that selection of the aggregate can be based upon considerations of its effect on thermal properties of concrete, it would be desirable to choose aggregate capable of producing a concrete which would heat or cool most readily to the temperature of its surroundings. The value of the diffusivity is an index of the facility with which concrete will undergo temperature change. Average values of the diffusivity in feet squared per hour for concrete containing various types of coarse aggregates are as follows:

Rock type of coarse aggregate	Diffusivity of concrete sq. ft. per hr.
Quartzite	0.058
Limestone	0.051
Dolomite	0.050
Granite	0.043
Rhyolite	0.035
Basalt	0.032

It is seen that variations of rock type of only the coarse aggregate can cause variations in diffusivity of almost 100 percent.

d. Thermal characteristics of concrete—The thermal characteristics of concrete must usually be accepted and cannot be selected or controlled. However, it is necessary that the thermal properties be determined if estimate is to be made of the temperature behavior of the concrete. The value of the diffusivity (h^2) is given by the relation

$$h^2 = \frac{K}{C\,\rho} \qquad (1)$$

where K is the conductivity, C is the specific heat, and ρ is the weight per unit volume of the concrete. The diffusivity may be determined by observing the flow of heat through a test specimen and computing the average diffusivity; or by determining the conductivity, specific heat, and unit weight, separately, and computing the diffusivity from these values. The latter method permits the determination of the variation of the thermal properties and the diffusivity with temperature of

the concrete. Ordinarily an average value of the diffusivity over the temperature range to which the concrete is subjected is used as a diffusion constant, and the error introduced by this approximation is certainly within the error of the estimated exposure conditions. Values of the diffusion constant may be estimated very closely if the percentages of the various ingredients and the mineral composition of fine and coarse aggregates are known. Empirical factors of the contribution of each percent by weight of the various concrete materials on the thermal properties of concrete have been obtained from laboratory investigations.[2] Thermal properties of a large number of specimens representing concrete from some twenty different dams have been determined in the laboratory of the U. S. Bureau of Reclamation. At 70 F., values of the diffusion constant vary from 0.024 sq. ft. per hr. for concrete from Bull Run Dam to 0.060 sq. ft. per hr. for concrete from Seminoe Dam. These comparative values indicate that it would require 2.5 times as long, under the same conditions, to cool the concrete of Bull Run Dam as to cool Seminoe Dam. Values of 0.040 sq. ft. per hr. or 1.0 sq. ft. per day are close to the average of all specimens tested and may be used for rough estimates when more definite information is lacking.

e. Physical properties of concrete—The development of a concrete highly resistant to cracking by selection of materials producing favorable physical properties offers a possibility that has not been adequately explored. Factors which must be considered in determining the relative resistance of various concretes to temperature-induced cracking are the temperature change, thermal coefficient of contraction and expansion, modulus of elasticity, creep characteristics, and the tensile strength. These are recognized[3] as important properties of concrete, but attention is usually concentrated on the effect of a single one of these factors rather than to the combined effect of all of them. For instance, low-heat cement (A.S.T.M. Type IV) was especially developed to reduce the cracking resulting from the large relative temperature changes that accompany the use of normal cement (A.S.T.M. Type I) in massive structures. The advantage of low-heat cement compared with normal cement was originally based on the comparison of their heat-generating characteristics. A later comparison, made in objecting to consideration of the heat-developing characteristic by itself, was based upon the strength-heat ratio of the different concretes. When this ratio of two factors is considered, the conclusion may be reached that concrete containing normal cement is more resistant to temperature cracking than concrete made with low-heat cement. Actual behavior in structures does not verify such conclusion. The modulus of elasticity and creep characteristics play an important role in the cracking tendency of different concretes.

(3) See references at end of paper.

This is indicated in the relative behavior of two large straight gravity dams of similar size and comparable exposure conditions which were constructed within recent years under similar careful control of construction operations. Very little cracking was experienced in the concrete of the Hiwassee Dam which had an abnormally low modulus of elasticity compared with its strength and a relatively high creep coefficient. The concrete of Norris Dam, which had normal elastic and creep characteristics, was cracked quite extensively. At Hiwassee Dam a great deal of attention was given to controlling concrete temperatures with the result that they were apparently more favorable than at Norris Dam. However, the marked difference in crack development could not be accounted for by the differences in temperature conditions alone. To a considerable extent they must be attributed to favorable physical properties of the concrete obtaining at the time critical temperature changes were taking place.

3. Requirements of design and construction

a. Size and shape of section.—Where temperature stresses are controlled and assurance is felt that the section will be free from cracks, higher load stresses might be allowable and the dimensions of the section reduced accordingly. In other words, the "factor of uncertainty" might be lowered as more accurate estimates of temperatures and thermal stresses are made available and as more positive control is developed. The dimensions of the structure determine largely the time required for the concrete to reach temperature equilibrium without artificial cooling. Except for losses from the joint and construction surfaces during the construction period, all excess heat ordinarily must be lost through the boundaries of the section. Some of the problems of temperature determination arising primarily from the size and shape of the section, with solutions given or referred to, are presented in section 6.

b. Contraction joints—*(1) Spacing of joints*—The spacing of contraction joints and the temperature control are interdependent to a large extent. With ordinary present construction, contraction joints or "formed cracks" must be provided at frequent intervals if cracks are to be avoided. Experience has indicated that with mass concrete the spacing between contraction joints may seldom exceed 50 feet, and usually a closer spacing is considered desirable. Exceedingly large sections thus require contraction joints in two directions, as was considered necessary for Boulder, Grand Coulee, and Shasta Dams. A similar design for Friant Dam had been contemplated before it was determined that more definite control of the placing temperature and of the temperature rise would enable the longitudinal joints to be eliminated, which would effect a material reduction in the construction costs.

In controlling the temperature to permit wider than normal spacing of contraction joints, the maximum temperature must be held low enough so that contraction of the concrete with subsequent temperature decline will not cause excessive tensile stress in the restrained mass. If the concrete could be placed at a sufficiently low temperature and the temperature rise so controlled that the maximum temperature attained would not exceed the ultimate temperature, the limit of joint spacing would depend not on temperature considerations but only on such physical factors as mixing plant capacity or limitations in handling or placing concrete.

The 50-foot spacing of transverse joints for Friant Dam was retained largely to facilitate construction. If a 5-foot lift is to be placed over the whole area of a block without cold joints, the dimensions of the block must be small enough, in at least one direction, to permit the previously placed concrete always to be covered while it is still soft. With temperature control that would prevent cracks in the 260-foot, upstream-downstream dimension it was recognized that the opening of the transverse contraction joints spaced 50 feet apart would indeed be very small. Where these joints are exposed at the surface or by galleries crossing the joints, the openings were scarcely noticeable. The maximum opening indicated by joint meters embedded across the joints in the interior amounted only to 0.045 inches. It was considered desirable to grout the contraction joints of Friant Dam primarily to prevent leakage through the joints. With the uneven and very small openings that must have been obtained over part of the joint area, it is questionable whether the joints of Friant Dam were completely grouted over their entire surface. Positive grouting of the joints over the whole area is not important in the transverse joints of a straight gravity dam but would be important for the longitudinal joints of a dam of this type or for the radial joints of an arch dam. Wherever stress is to be carried across the joints, the spacing of the joints and the temperature control must be such that the joints will open sufficiently so that subsequently they can be grouted completely. The least joint opening that is considered to be readily groutable is 0.03 inches.

(2) *Grouting of joints*—Contraction-joint grouting with cementitious mixtures of relatively thin consistency, is largely a special requirement of mass concrete in order to assure its integration and action as a continuous mass after it has been brought to the desired temperature. The temperature control directly affects the joint opening and thus the time when the joints should be grouted. Conversely, the requirement for grouting the joints at a given time in order to place the structure in service may dictate the extent and nature of temperature control of the concrete. Requirement for grouting contraction joints is limited

almost entirely to massive structures such as dams. For this reason discussion of the practice as related to dams is given as an example of the methods and materials used.

In arch dams, contraction joints on radial lines divide the arch ring into sections which lose contact when the joints widen as the concrete ultimately shrinks. There can be no arch action until the faces of these radial joints are again brought into contact, either by the downstream deflection of the cantilever elements when the dam assumes its water load, or by filling the open joints with grout. The first alternative is undesirable because deflection of the cantilever elements produces excessive tension, and because of unequal stress distribution over the joint face as a result of incomplete contact due to uneven shrinkage in various parts of the structure. When the open joints are properly filled with hardened grout, the structure will act as a monolith in which the stresses and deflections are comparatively uniform and much more consistent with the design assumptions. Instead of grouted contraction joints, a method sometimes used involves the construction of blocks separated by slots which are filled with concrete at a time when the temperature condition of the blocks is most favorable. Lean concrete may be used in filling the slots in order to minimize contraction.

Since contraction-joint grouting materially reduces leakage, it is a desirable procedure for the transverse joints of straight gravity dams as well as for arch dams.

The systematic grouting of vertical contraction joints by means of an embedded pipe system is a comparatively recent development. According to available records, the Bullards Bar Dam (1924), the Cushman Dam No. 1 (1926), and the Pacoima Dam (1927) are among the early examples of dams grouted by this method.

As now developed, the embedded pipe and unit system discharges grout through circular outlet units, uniformly spaced over the face of the joint, as illustrated in Fig. 2. The outlet units are composed of two electrical-conduit box covers, which are called buttons when used for this purpose. The outlet buttons, connected to tees on vertical riser pipes, are attached to the concrete forms. When the forms are stripped, the cover buttons are attached with wire to form the complete outlet unit. The vertical riser pipes are connected to a horizontal supply header below. Instead of connecting the vertical risers to a horizontal header at the top, recent installations have included a separate horizontal outlet header connecting grout outlet units at the top of the joint to force grout to travel through the joint before escaping. This provides for the escape of air and excess water and gives more positive assurance of completely filling the joint. The area of the joint to be grouted by

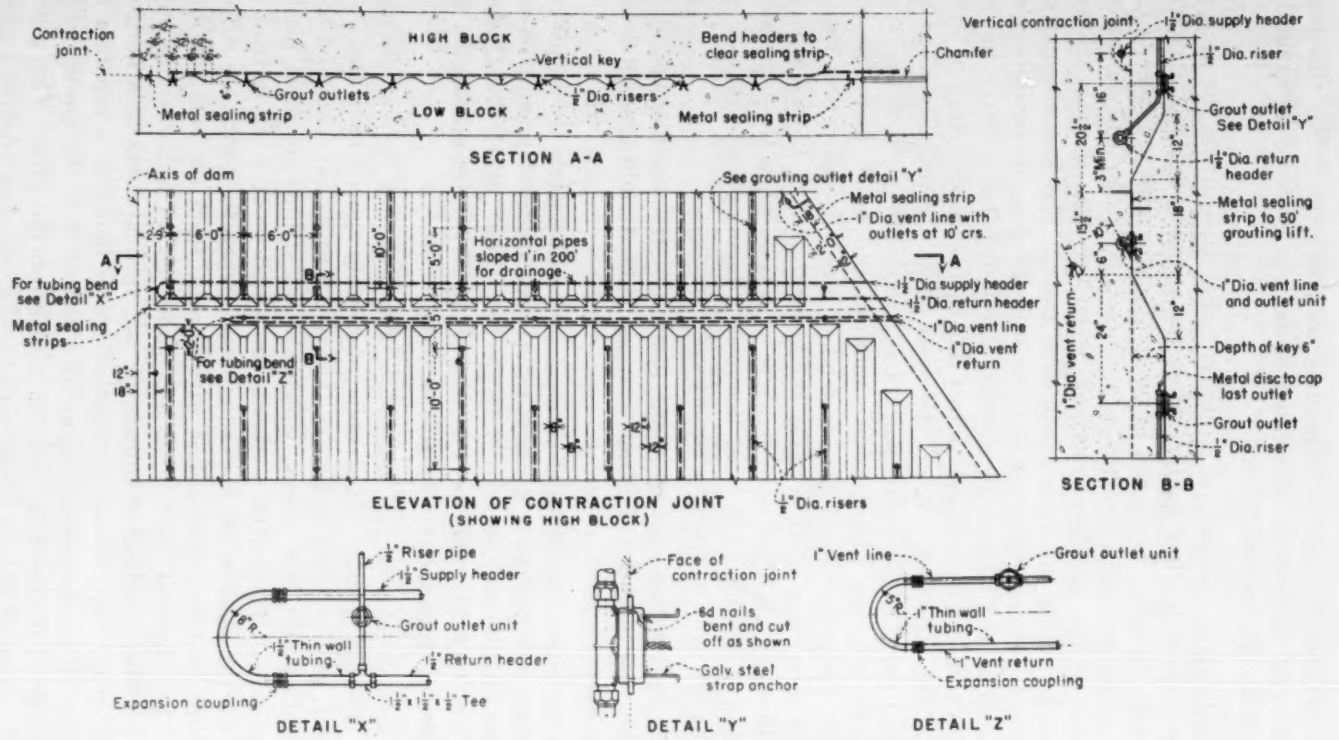


Fig. 2—Pipe and unit system for grouting contraction joints

each such pipe system is enclosed by a metal sealing strip extending across the joint opening. Grout injected into the lower header is forced up the risers and into the contraction joint through the outlet buttons attached to the risers while air and water escape through the outlet header. Intermittent injection of grout into the joint allows time during the rest intervals for equalization of the level of grout in the joint and for escape of air and water. When thick grout flows from the outlet header, injection is stopped for a while to allow the grout to settle. After several repetitions of showing of thick grout the header is capped. The pressure on the supply is then increased to the allowable limit for the particular joint so as to force the grout into the small openings of the joint and to drive the excess water into the pores of the concrete. This limiting pressure, usually not over 50 pounds per square inch, must be low enough to avoid deflecting the block excessively or causing a V-shaped opening of the grouted portion of the joint below. This maximum pressure is maintained until no more grout can be forced into the joint faces. The valve on the inlet header is then closed, and the supply piping is removed and cleaned before grout sets in the piping and pumps.

c. Forms and form removal—Wooden forms are fairly good insulators of heat, the conductivity of a one-inch thickness of wood being about that of one to two feet of concrete. Wooden forms thus prevent heat from being lost readily from the surface during the time that the forms are in place. Very often, forms are removed two to three days after placement of concrete which, with respect to sequence in construction operations, seems to be the best and most logical time. At that time, however, surface concrete is close to the maximum temperature. Cold-water curing is often started coincident with the removal of forms, when the temperature difference between concrete and water is a maximum. This subjects the surface to rapid temperature drop which may produce surface cracks. For optimum temperature conditions, the forms should be removed and curing started as early as possible, preferably within 24 hours; this will flatten the temperature gradient within the concrete, thus reducing the tendency toward the formation of cracks. If the forms cannot be removed during the very early ages, as is often the case, then it is better that they be left in place for at least a week or until other measures of control have reduced the temperature of the concrete well below the maximum, although such procedure may not provide the best early curing conditions. It has been shown [6] that adverse temperature conditions due to untimely removal of forms may be responsible for the start of a great deal of cracking that occurs in thick walls. An advantage would be obtained in most instances through the use of steel forms, which will not retard the loss of heat from the surface.

d. Curing with water—Curing of exposed surfaces is usually required to prevent loss of moisture from the concrete during the early period of hardening. Where curing takes the form of continued spraying with water the surface can be kept continually at the temperature of the water which may be considerably lower than the air temperature. Thus, water curing is advantageous over sealing-compound curing in the withdrawal of excess heat from the mass. However, the application of a suitable curing compound is to be preferred to intermittent sprinkling, which often results in surface drying between sprinklings. In a hot, dry climate, surface temperatures approaching wet bulb temperature, which may be as much as 30 degrees below air temperature, may readily be obtained by use of fog-spray curing. At Bartlett Dam in Arizona, where fog-spray curing was used during the summertime, curing temperatures close to 75 F. were maintained even during the warmest weather.

e. Special treatment to reduce placing temperatures—A form of temperature control that may be classed with the methods of construction involves special treatment of the concrete materials to reduce the placing temperature. This procedure has been discussed in some detail in section 1.

f. Artificial cooling—Artificial cooling is definitely established as the most effective means of controlling the temperature of concrete. Furthermore, the most satisfactory, most flexible, and for the degree of control obtained, the most economical method of artificially cooling the concrete is by circulating cold water through a pipe system embedded in the concrete.

(1) *Advantages*—The advantages of cooling the concrete artificially are:

(a) The concrete may be cooled to the final stable temperature in a month or two during the period of construction, after which, with the concrete at minimum volume and the contraction joints opened to the fullest extent, the joints may be grouted, thus obtaining the monolithic structure assumed in design.

(b) The mass may be cooled uniformly, thus preventing large temperature gradients from the warm interior to the cold exterior which are conducive to cracking of the concrete at the exposed surfaces.

(c) Subcooling may be accomplished if desired, and in the case of an arch dam with joints grouted in the subcooled condition, the effect of the subsequent temperature rise will counteract and prevent excessive deflection due to water load.

(d) The temperature of the various parts of the structure may be controlled at the time of grouting the joints so that the effect of subsequent temperature change can be used to counteract unfavorable load-stress distributions.

(e) Artificial cooling started immediately after placing the concrete, and especially with the use of a cement which develops heat slowly, will lower the maximum temperature of the concrete considerably.

(f) With the temperature strains and resulting stresses imposed on the concrete in the early age, when the concrete is relatively plastic, greater relief of stress is obtained through plastic flow or creep than would be obtained at later ages.

Cooling with embedded pipes has the following advantages:

(a) Flexibility of arrangement—Cooling may be accomplished from either face or from galleries or shafts within the dam.

(b) Complete coverage—All parts of the dam may be cooled without difficulty.

(c) Flexibility of operation—Any desired degree of cooling may be accomplished, at any place, at any time.

(d) Low cost—Complete cost of actual installations in six dams ranging in size from 175,000 to 9,840,000 cubic yards, including all labor and material costs chargeable to cooling, varied from $0.13 to $0.55 per cubic yard of concrete cooled. It is likely that the unit cost of cooling much smaller masses would be somewhat higher.

(2) *Disadvantages and limitations*—The cost of artificial cooling is certainly its chief disadvantage, although this has been shown to be low. If the temperature of concrete is kept very low during early ages, and particularly with low-heat cement, the development of strength may be retarded materially, which may have to be considered in the design of form anchors or in the time of removal of forms. The chief limitation is that the rate of cooling with the embedded pipe system is much slower than the rate of heat generation at the early ages so that unless the pipes are spaced very closely the concrete temperature will rise materially above the initial temperature even though cooling is started immediately. It is not always possible to design a simple cooling system layout, so that costs may vary considerably on different structures. Pipes may be damaged or plugged during embedment. Silty water may clog the pipes. All such difficulties, however, can be overcome with care in installation and operation.

(3) *Description of embedded pipe cooling system*—The cooling system in most general use is made up of a number of embedded coils in parallel, served by supply and return headers. The coils are of one-inch outside diameter, thin-wall metal tubing about 800 ft. long, coupled with expansion-type couplings. The tubing is placed on the top of each lift of concrete after it has hardened. For the customary 5-ft. lift the vertical spacing of pipes is thus 5 ft. The horizontal spacing is varied to suit the purpose and is usually between 2 ft. and 6 ft. The velocity of flow of the cooling water through the embedded coils is not less than 2 ft. per second which amounts to about 4 gal. per min. in the 1-in. tubing. The water is usually pumped through the coils, though a gravity system may sometimes be used to advantage. When river water is used, the warmed water, after passing through the coils, is wasted. When using refrigerated water a closed system of piping is usually employed in which the warmed water is returned to the water coolers in the refrigerating plant and is used repeatedly. With a closed system the pumping head required is only that needed to overcome friction. The temperature of the concrete, for control of the cooling system operation, is determined by resistance thermometers embedded in the concrete or inserted in

pipes embedded in the concrete for that purpose. Descriptions of the application of embedded pipe cooling systems in control of temperature of several large dams are contained in the literature[11, 12, 15]. Methods of computing the average temperature of the concrete, the temperature distributions around cooling pipes, and the temperature rise of the circulating water under artificial cooling are included in section 6.

(4) *Control of rate of cooling*—Factors in the control of rate of cooling and of the total time required are the size, spacing, and length of the embedded coils, the rate of circulation through each coil, the thermal characteristics of the concrete, and the percentage of the total temperature differential between the warm concrete and the cold water that is required to be removed. The economy of cooling is also dependent on these same factors. The cooling rate is not greatly influenced by an increase in cooling pipe size, by shortening the embedded pipe length between inlet and outlet ends, or by increasing the supply of cooling water. Computations of the effect of increasing the rate of flow through the 1-in. pipes indicate a marked improvement of performance by an increase in flow up to about 4 gal. per min. Doubling the flow above 4 gal. per min. only decreases the time required for cooling some 20 to 25 percent for average conditions, whereas it doubles the capacity requirements, increases the friction losses, and more than doubles the power costs. The cooling rate may be effectively increased by decreasing the temperature of the cooling water or by adopting a closer pipe spacing.

g. Other expedients to control temperature—A method often considered because it seems to offer satisfactory temperature control involves the use of precast concrete blocks. The cost of double handling of the concrete and the difficulty in obtaining satisfactory joints usually militate against the procedure. If the entire structure is to be made of precast blocks, the joint material may occupy such a large percentage of the volume that satisfactory temperature conditions cannot be obtained. A possible economical adaptation to the construction of moderate-sized dams might be the use of precast concrete blocks on the upstream and downstream faces of the dam with lean concrete in very shallow lifts placed without joints in the interior. Thus the temperature rise of the interior might be limited to a very small amount, and durable surfaces might be obtained with the entire elimination of forms and prevention of the undesirable surface effects of large temperature variations which accompany concrete cast in place.

A suggested method involves the use of precast blocks or slabs for the exterior with the interior filled with coarse aggregate, and the whole mass solidified by injecting cement mortar into the spaces between the coarse aggregate. It is feasible with this method to cool the aggregate

in place by circulation of cold water through the packed aggregate and to grout the mass at the desired low temperature.

4. Rate of construction

a. Thickness of lifts—A number of other factors vie with temperature control to determine the most satisfactory thickness of lift for mass concrete construction. The present, most-commonly-adopted thickness of 5 ft. is a compromise that has proven to be very satisfactory. Under the usual conditions, where the temperature of the concrete at the time of placing is approximately at the average exposure temperature, close to one-third of the total heat developed by hydration of the cement may be lost from the top surface if the lift is exposed five days before the next lift is placed. Without cooling, the temperature rise near the base of the lift is very close to the adiabatic temperature rise during the first few days. Effective reduction in maximum temperature with some lowering of the temperature at the contact surface has been obtained using 2½-foot lifts in certain locations throughout the dam and in the base of Hiwassee Dam[9]. Favorable temperature distribution through the lift may also be obtained with the use of closely spaced cooling pipes at the bottom of the lift.

For thin sections, such as buttresses or walls, lifts of greater depth can be used without corresponding temperature rise because of loss of excess heat from the sides. Where the depth of lift is more than one-third the thickness of section, any further increase in depth of lift has no appreciable effect on raising the maximum temperature.

b. Time interval between lifts—Limitation of minimum time of exposure of the top of lift goes hand-in-hand with limitation of depth of lift for control of the temperature rise. Many specifications, therefore, limit the maximum depth of lift for large concrete dams to five feet and the maximum rate of placement to one lift in 72 hours. It is also desirable to place a limitation on the maximum time that a lift is to be exposed if exposure conditions widely different from those at time of placement are to be averted. Excessive cooling of the top of the lift promotes the formation of cracks extending downward from the surface. Where cracks were observed across the tops of several lifts in Friant Dam, they first appeared at ages from 12 to 18 days after placement of the lift. From the viewpoint of temperature control the most favorable time for covering the top of a five-foot lift seems to be from 5 to 10 days after the lift has been cast.

c. Time of exposure at contraction joints—If cracks are to be averted in the sides of blocks, the time of exposure at contraction joints should be limited to the minimum possible. Unless the blocks are very small the loss of heat to the sides has little effect on the maximum temperature.

Construction and handling of the forms often determine the minimum acceptable height differential between adjacent blocks. Diversion and care of the river sometimes require that the sides of certain blocks be exposed for unusually long periods. Special treatment, as at Hiwassee Dam[9], is beneficial in such cases. Such blocks were provided with cooling coils and a gradual temperature gradient was established from the exposed face to the interior. Time of removal of forms, which has been discussed in section 3c, has a marked effect on the cracking of the sides of blocks during the time that the sides are exposed.

d. Seasonal limitations for placing concrete—It is desirable, from the temperature standpoint, that mass concrete be placed at the lowest possible temperature consistent with insurance against freezing. Conversely, it is undesirable to place concrete in very hot weather. The worst temperature stress conditions are likely to occur in concrete placed in the hot weather of late summer which is soon thereafter subjected to the rapidly dropping temperatures of early fall. Construction of large structures usually extends over several seasons and sometimes over several years. To stop placement entirely during the hottest time of the year is seldom feasible in such cases, and other measures of control in addition to artificial cooling can be exercised to circumvent the ill effects of hot-weather placement, such as the precooling of aggregates and mixing water or the use of shallow lifts with a longer interval between lifts.

5. Character of foundation

a. Temperature—Restraint to volume change in the concrete is quite high at the foundation contact. For best results the concrete temperature, at time of placement, should be at or below the rock temperature, and increases in temperature in both concrete and rock should be kept to a minimum. Such changes as do occur should be made to take place slowly over a long period of time. High tensile stresses are induced in concrete placed hot upon cold rock when the concrete temperature is subsequently lowered. Fissures and cracks exist in nearly all rock foundations. When the rock is hot, such fissures are closed. Cracks will form in concrete placed over such fissures if the rock temperatures are subsequently lowered and the fissures opened. At Friant Dam cooling pipes at 2½-foot spacing were placed on the rock, and cooling water about 10 degrees lower than concrete placing temperature was circulated through these pipes at the time the concrete was placed, in an effort to prevent changes in temperature of the rock due to placement of concrete. Observations of temperatures at various depths in the foundation rock indicated a maximum temperature rise of 6 degrees F. one foot below the surface of the contact between rock and concrete; this maximum occurred 5 days after the concrete had been placed.

b. Thermal characteristics—Even though the thermal characteristics of the rock be unfavorable, it is not likely that difficulty from that source will be experienced. Certain rocks have extremely low thermal coefficients of expansion and contraction, and it is conceivable that if concrete were cast upon such rock and permitted to undergo a large temperature change, the bond between concrete and rock would be broken. If the contact temperatures are prevented from changing materially, differences in thermal characteristics between rock and concrete are not important.

c. Profile and preparation—The subject of foundation treatment and preparation is one that, in principal part, is extraneous to this report. Such treatment includes excavation to sound rock; systematic drilling of the foundation; and subsequent grouting of seams and fissures to consolidate the foundation rock, to reduce seepage by increasing percolation distances, and to reduce uplift pressure. The treatment is discussed herein only insofar as it affects the behavior of mass concrete placed upon a rock foundation.

Restraint of the foundation to reduction in volume of the concrete is the source of much cracking. These cracks usually begin at irregularities in the foundation, which are points of stress concentrations in the lift placed directly on the foundation, and progress upward as construction proceeds. Sharp breaks in the foundation and large, abrupt differences in elevation are particularly undesirable. Where a large difference in elevation exists, characterized by a sharp break and a level bench at the upper elevation, cracks ordinarily start at the break in the foundation and progress upward. These cracks are due to differential vertical contraction. They may be averted by raising the concrete to the bench level and cooling completely all concrete below the level of the bench before placing the next lift of concrete which will then rest partly on rock and partly on concrete. Very often, a small amount of additional excavation to remove sharp angular projections and irregularities or to slope vertical breaks to the nearest contraction joint or boundary would be the most efficacious treatment that could possible be prescribed and might even prove to be the least expensive.

6. Temperature computations

a. Cooling of a flat slab—Loss of heat from both faces of a dam or loss from the sides of a wall or buttress may be likened to the loss from a flat slab exposed on both sides. Solutions of the problem of cooling of a flat slab losing heat only from the sides have been published[7] and are repeated here in part. The relative time required for loss of a given percentage of excess heat from slabs of different diffusion constant and different thickness is given by the relation

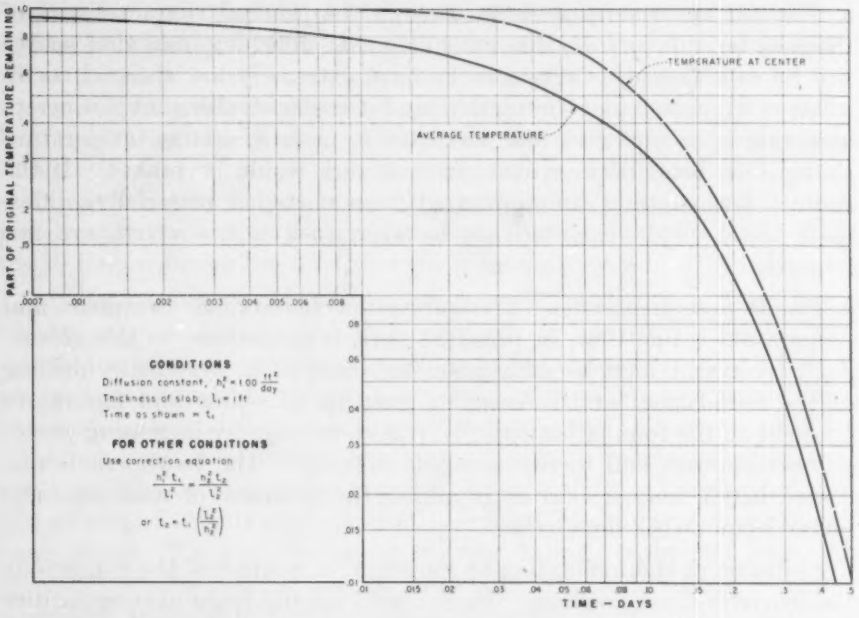


Fig. 3—Cooling of flat slabs exposed on two sides, uniform initial temperature distribution

$$\frac{h_1^2\, t_1}{l_1^2} = \frac{h_2^2\, t_2}{l_2^2} \qquad (2)$$

where h^2 equals the diffusion constant, t equals time, and l equals the thickness, all in consistent units; and the subscripts denote the instances 1 and 2. In other words, the time varies inversely as the diffusion constant, as has been indicated previously; and also directly as the square of the thickness. Thus, a concrete wall 5 ft. thick may lose 90 percent of its excess heat in 5 days whereas to lose the same percentage of excess heat from a dam 50 ft. thick will require 500 days, or roughly 1.4 years, and from a dam 200 feet thick will require 8,000 days, or 22 years. With the above relationship the solid curve of Fig. 3 may be used to compute the mean temperature of any slab at any time. The curve shows directly the relationship when h^2 and l are unity. For given conditions equation (2) is used to compute equivalent values for use with the curve.

In the same manner as the mean temperature of the slab is determined with the solid curve of Fig. 3, the dashed curve may be used to determine the temperature at the center of the slab. For example: A dam at a certain elevation is 40 ft. thick. If it is exposed to constant temperature on both sides, how long will be required for 60 percent of the excess heat to be lost? What will be the temperature at the middle of the section

at that time if the exposure temperature is 55 F. and the temperature of the concrete at the start was 105 F.? Assume h^2 for the concrete to be $0.050 \frac{\text{ft.}^2}{\text{hr.}}$ or $1.20 \frac{\text{ft.}^2}{\text{day}}$. If 60 percent of excess heat has been lost, then 40 percent or 0.40 remains. From the curve of average temperature, the time required for a slab one foot thick, $h^2 = 1.00 \frac{\text{ft.}^2}{\text{day}}$, to lose 60 percent is 0.07 days. The time required for a 40-foot thickness of the given concrete is by equation (2),

$$t_2 = \frac{1.00}{1.20} \times \frac{40^2}{1.00^2} \times 0.07 = 93 \text{ days or } 3.1 \text{ months.}$$

At that time 0.63 parts of excess heat will still be remaining at the center of the section. The excess heat was originally

$$105° - 55° = 50°$$

$0.63 \times 50 = 31.5$ degrees above the exposure temperature still remaining.

The temperature at the center is then

$$55 + 31.5 = 86.5 \text{ F.}$$

With the same relationship of given values of h^2, t, and l, with unit values of these variables, the curves of Fig. 4 may be used to determine the temperature at any point in the slab or the temperature distribution throughout the slab at any time. Both Fig. 3 and Fig. 4 are based upon the idealized conditions of uniform initial temperature of concrete with the surfaces maintained at placing temperature. Furthermore, the simplifying assumption is made that surface temperatures are equal to the exposure temperatures. For very thin slabs this assumption will introduce appreciable error, but it is generally permissible to neglect the effect of the coefficient of heat transmission from concrete to air for all mass-concrete structures.

b. Cooling of a prism—The preceding solutions may be extended to the case of a prismatic body of rectangular cross section having uniform initial temperature and exposed on four sides to zero temperature. The part of the original heat remaining in a prism may be computed by finding the part remaining in two slabs of respective thickness equal to the dimensions of the prism, and multiplying the two quantities so obtained together. This procedure may be applied with Fig. 3 to obtain the average temperature of the prism or with Fig. 4 to obtain the temperature at any point in the prism. The application may be made to a rectangular pier or to a block or panel of a dam exposed on four sides.

c. Cooling of a parallelopiped—The solutions for cooling of a slab may be further extended to the case of a rectangular parallelopiped of uni-

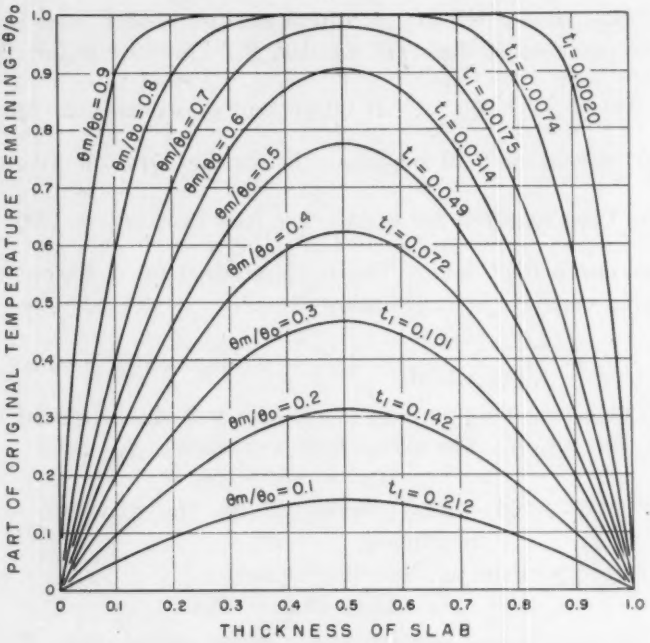


Fig. 4—Temperature distribution in flat slabs exposed on two sides

CONDITIONS

Uniform initial temperature distribution.
Diffusion constant = $h^2 = 1.0 \frac{ft^2}{day}$
Thickness = $l_1 = 1.0$ foot

NOTATION

θ = Temperature at time given
θ_0 = Initial temperature
θ_m = Mean temperature of slab
t_1 = Time in days

For other conditions, $\frac{h_1^2 t_1}{l_1^2} = \frac{h_2^2 t_2}{l_2^2}$

form temperature and exposed on the six faces to zero temperature. The part remaining in this case can be obtained in a manner similar to that described above, by finding the part remaining for each of the three dimensions of the parallelopiped and multiplying the three parts remaining together. Application might be to the cooling of precast concrete blocks used in dam construction. If one side of a slab is perfectly insulated, the slab will cool as though it were part of a slab of twice the thickness exposed on both sides. Similar statements hold for the prism and the parallelopiped.

d. Slabs exposed to variable temperatures—The solutions of this problem find application in estimating the temperature changes to which dams are subjected because of the variation of temperatures of the air and water with which the surfaces of the dam are in contact. Of particular interest are the reduction of temperature variation with distance from the exposed surface, the variation of mean temperature of the dam, and the lag of the mean temperature of the dam with respect to the external

temperature variation. Principal cycles of variation of external air temperature, with amplitudes readily determinable from records published by the Weather Bureau, are the daily cycle and the annual cycle. Limits of the observed temperature variation can usually be expressed by inclusion of one or more additional terms whose amplitude and length of cycle are chosen to fit the observed data. Addition of a single 15-day cycle has been found to represent conditions suitably close for many locations. Thus the exposure conditions are represented by a limited number of sine waves of temperature.

(1) *Reduction of temperature variation with distance from exposed surface.* With the temperature of the surface represented by a sine wave and with the slab sufficiently thick so that the temperature variation is negligible at the opposite side to that exposed, the range of temperature variation at any distance from the surface can be computed by the relation

$$\frac{R_X}{R_O} = e^{-\frac{x}{h}\sqrt{\frac{\pi}{\gamma}}} \qquad \ldots\ldots(3)$$

where

R_X is the range at distance x from the surface,
R_O is the range at the surface ($x = 0$),
h is the square root of the diffusion constant,
γ is the length of the cycle of temperature variation.

With the diffusion constant h^2 expressed in the units $\frac{\text{ft.}^2}{\text{day}}$, the units of x will be feet and the units of γ will be days. Inspection of the exponent of e discloses that for equal values of R at different points x_1 and x_2 there will exist a relationship between the other variables of the form

$$x_2 = \sqrt{\frac{\gamma_2}{\gamma_1}}\, x_1 \qquad \ldots\ldots(4)$$

The following example shows the computation for the range of temperature variation at various depths when the surface is subjected to a 15-day cycle of variation and the diffusion constant is $h^2 = 1.00\frac{\text{ft.}^2}{\text{day}}$. By use of equation (4), corresponding depths for the annual variation are given by

$$x_2 = \sqrt{\frac{365}{15}}\, x_1 = 4.93x_1$$

For daily variation

$$x_2 = \sqrt{\frac{1}{15}}\, x_1 = 0.258x_1$$

The computations are shown in Table 1 and the results are plotted in Fig. 5.

TABLE 1—TEMPERATURE VARIATIONS WITH DEPTH IN CONCRETE

x (ft.)	$\frac{x}{h}\sqrt{\frac{\pi}{\gamma}}$	$\frac{R_x}{R_o} = e^{-\frac{x}{h}\sqrt{\frac{\pi}{\gamma}}}$	Equivalent Depths	
			Daily variation	Annual variation
0	0	0	0	0
1	0.46	0.63	0.258	4.93
2	0.92	0.40	0.516	9.86
3	1.38	0.25	0.774	14.8
4	1.84	0.16	1.03	19.7
5	2.30	0.100	1.29	24.6
6	2.76	0.063	1.55	29.6
7	3.22	0.040	1.81	34.5
8	3.68	0.025	2.06	39.4
9	4.14	0.016	2.32	44.4
10	4.60	0.0100	2.58	49.3
11	5.06	0.0063	2.84	54.2
12	5.52	0.0040	3.10	59.2
13	5.98	0.0025	3.35	64.1
14	6.44	0.0016	3.61	69.0

(2) *Time lag in penetration of surface variation.* The phase of the temperature wave is progressively delayed with increasing depth. The time lag of a sinusoidal variation at any depth may be obtained from the relation

$$t = x\sqrt{\frac{\gamma}{4h^2\,\pi}} \qquad \ldots\ldots(5)$$

where t is the time in units consistent with the other variables. Of interest in certain instances is the depth at which the lag is one complete period behind the surface temperature. This distance λ, termed the "wave length," which is the distance between points at which maxima and minima of temperature occur simultaneously, is related to the terms in the above equation as follows:

$$h^2 = \frac{\lambda^2}{4\,\pi\,\gamma} \qquad \ldots\ldots(6)$$

or

$$\lambda^2 = 4\,\pi\,h^2\,\gamma$$

(3) *Variation of mean temperature.* Computations of variation of the mean temperature of a slab exposed on both sides to sinusoidal temperature variations may be made with the use of the curve of Fig. 6. The diagram shows directly the ratio of the variation of mean temperature of the slab to the variation of external temperature for slabs of different thicknesses in feet l having a diffusion constant h_2 of 1 sq. ft. per

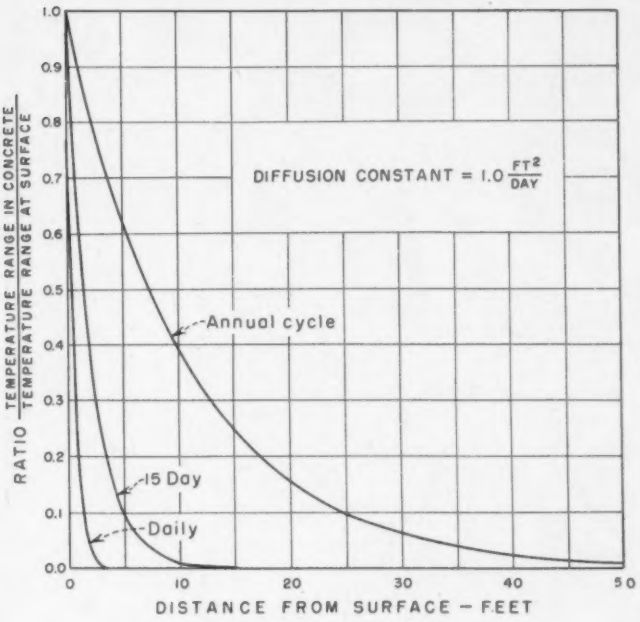


Fig. 5—Temperature variation with depth in concrete

day when subjected to a sinusoidal temperature change with a period γ of 1 day. Correlation to any other case is made through the use of the correlation equation.

$$\frac{h_1^2 \gamma_1}{l_1^2} = \frac{h_2^2 \gamma_2}{l_2^2} \quad \ldots\ldots\ldots\

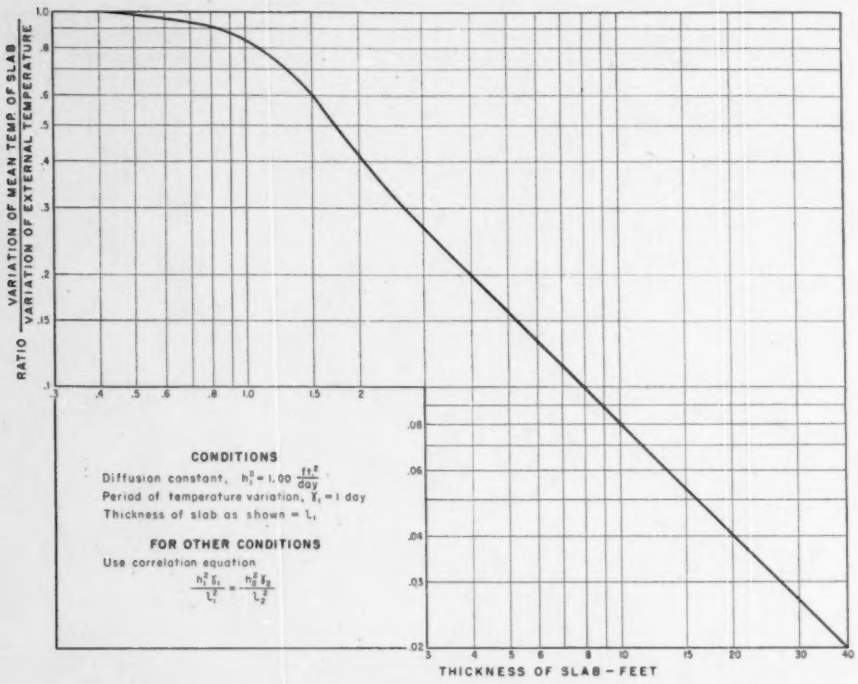


Fig. 6—Temperature variations of flat slabs exposed to sinusoidal temperature variations on both faces

perature toward the annual temperature which is not affected by the external temperature fluctuations, and then to superimpose upon those results the changes due to the cyclic fluctuations.

e. Stepwise computation of cooling of a flat slab—The temperature difference between the interior and the surface of the concrete is affected by the heat generated during hydration of the cement as well as by the variation of the surface temperature. The effects of these variations in temperature differences can be evaluated by dividing the total time into intervals, during each of which the temperature difference is considered to be constant. The effect of each temperature difference is carried throughout the remainder of the total time from the time in which it is first operative. The average temperature (or center temperature) at any given time is determined by summation of the effects of the individual temperature differences. A stepwise computation as shown in Table 2 permits the orderly arrangement and ready determination of the temperature of the slab at any time. In the following example the average temperature at the end of each of the first seven days is determined for a 5-ft. thick wall containing 1.25 barrels per cubic yard of low-heat

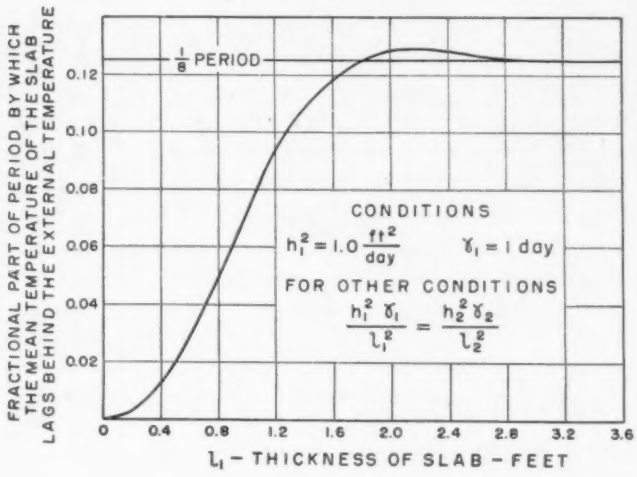


Fig. 7—Lag of mean temperature of slab exposed to sine wave of temperature variation on both faces

cement having a diffusion constant of $0.050 \frac{\text{ft.}^2}{\text{hour}}$ or $1.20 \frac{\text{ft.}^2}{\text{day}}$ and heat-developing characteristics as shown in Fig. 1, with an initial temperature of 60 F., and with the surface at 60 F. during the first day and dropping two degrees each day thereafter.

f. Tabular methods of computing temperatures—Two simple and very useful procedures for the computation of temperature distributions in mass concrete under any given conditions by step-by-step processes have been developed, one by Schmidt[14] and the other by Carlson[4]. Both methods consist in dividing the concrete into space elements and computing first the temperature after the lapse of one time interval, then after another, and so forth. Although the methods are approximations, any degree of accuracy desired can be obtained by selection of space and time intervals commensurate with the accuracy desired. To make the intervals smaller than necessary, however, only increases the work needlessly.

(1) *Schmidt's method.* In application of Schmidt's method[14] the body is divided into intervals which, in the direction of heat flow and with respect to the time intervals chosen, have the relation

$$\Delta t = \frac{(\Delta x)^2}{2 h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where

Δt is the time interval,
Δx is the length of element or space interval,
h^2 is the diffusion constant.

TABLE 2—SAMPLE COMPUTATION OF COOLING OF A CONCRETE WALL 5 FT. THICK

Time t_2 days	Temp. diff. Increment of rise plus change in exposure	Temp. at end of interval °F	Equivalent time t_1 from equation (3) $t_1 = 0.048\ t_2$	Parts remaining from Fig. 3	Average temperature rise (°F) at end of each interval of time (days)						
					1	2	3	4	5	6	7
0		60									
1	16.9	60	0.048	0.50	8.5	5.4	3.2	2.0	1.3	0.8	0.5
					8.5						
2	17.0	58	0.096	0.32		8.5	5.5	3.2	2.0	1.3	0.8
						13.9					
3	8.9	56	0.144	0.19			4.5	2.9	1.7	1.1	0.7
							13.2				
4	5.1	54	0.192	0.12				2.6	1.6	1.0	0.6
								10.7			
5	3.9	52	0.240	0.077					2.0	1.2	0.7
									8.6		
6	3.6	50	0.288	0.048						1.8	1.2
										7.2	
7	3.4	48	0.336	0.029							1.7
											6.2
Exposure temperature at end of interval °F					60.0	58.0	56.0	54.0	52.0	50.0	48.0
Average temperature of concrete at end of each day °F					68.5	71.9	69.2	64.7	60.6	57.2	54.2

The units of Δt and Δx must be consistent with the units in which the diffusion constant is expressed. In the case of one-dimensional heat flow the rate of temperature change of any interior element depends only on its own temperature and on that of the two adjoining elements; and with the space and time intervals chosen in accordance with equation (8), the temperature relationship within each group of three elements is such that the temperature of the middle element at the end of the time interval is approximately the average of the temperatures of the elements on either side at the beginning of the time interval. In symbol expression, if θ_p, θ_q, and θ_r are the temperatures of three successive points at time t, then at time $t + \Delta t$

$$\theta_q + \Delta \theta_q = \frac{1}{2}(\theta_p + \theta_r) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The computation procedure consists merely in writing as a column or row of figures the initial temperature distribution and averaging alternate terms to obtain a second column representing the temperature at the end of the first time interval, then averaging alternate terms of this second column to obtain a third column, and so on. The boundary conditions determine the end figures of the columns. If heat is being generated, the amount generated during each interval, expressed as degrees of adiabatic temperature rise, may be added after averaging. Schmidt's method lends itself readily to a graphic solution which is often found to be convenient.

(2) *Carlson's method.* The method developed by Carlson[4] is similar in principle to that described above. The body is divided into space intervals and the temperature of each element is computed after the lapse of one time interval, and so on. Although solution involves solving as many simultaneous equations as there are elements chosen, never more than three unknowns appear in any one equation, and procedure for solution simply involves filling three columns of a table by simple arithmetic processes to obtain the temperature for each element at the end of the time interval in the third column.

For the same sized intervals, Carlson's method is more accurate than Schmidt's method, but the latter is so much simpler and computation proceeds so rapidly for a given ease that smaller intervals can be used with Schmidt's method and increased accuracy obtained on most types of problems with less work than with Carlson's method. Both methods are adaptable to solutions of flow of heat in more than one direction and to a large variety of problems.

g. Computations of average temperatures with artificial cooling—For purposes of computation it is assumed that each pipe cools a cylinder of concrete having a cross section equal in area to the cross-sectional area

Fig. 8—Mean temperature of concrete

served by each pipe. Curves of rigorous solution of cooling of hollow cylinders have been published[7, 13]. More convenient curves of nomographic form, expressed in the units of most common usage, have been developed from those basic curves. Figures 8, 9, and 10 are curves of this nature used in estimating temperatures obtained and plant capacities required under given conditions. Figure 8 shows the variation of mean temperature with time, for the concrete over the whole length of the cooling coil, for varying conditions of coil length, flow of water, horizontal spacing of pipes, and diffusion constant. The curves are based upon the use of 1-inch outside-diameter cooling pipes placed 5 ft. apart vertically. Similarly, Fig. 9 gives the variation of mean temperature with time, for the concrete at any given length from the inlet, for the same variable conditions. With a long, embedded cooling coil the temperature of the water flowing through the coil is raised materially so that the effective cooling is much less at the outlet end than at the inlet end. With water flowing in the same direction continually, Fig. 9 permits determination of the variation of mean temperatures along the length of the coil. In practice, the headers serving the embedded coils are usually cross connected or the hose connections from the inlet and the outlet headers to the individual coils are manipulated so that the flow of water may be reversed at any time, the principal advantage being that the pipes can thus be kept from clogging, even with the use of turbid water. Figure 10 indicates the temperature rise of the water through the coil at any time after cooling is started under the same variable conditions given in Figs. 8 and 9. This value is important for determination of plant requirements if refrigerated water is to be used for cooling. All temperature rises indicated by curves of Figs. 8, 9, and 10 are given in terms of proportion of the difference between the concrete temperature at the start of cooling and the inlet water temperature. It is assumed that the inlet temperature of the water is constant and that there is no heat generated in the concrete during the cooling period. If the concrete gains heat or the water temperature varies, the curves may still be used in a stepwise computation such as described previously for cooling of a flat slab.

h. Temperature distribution around cooling pipe—The temperature distribution surrounding a cooling pipe during the cooling period may be computed by the use of Fig. 11. The curves show directly the values obtained at time t_1 when the diameter of the cooled cylinder D_1 is 10 feet and the diffusivity h_1^2 equals 1.0 ft.^{2}/day, and when the ratio of outer radius to inner radius of the cooled cylinder $\frac{b}{a}$ equals 100. Correlation with an actual case is made by means of the correlation formula

KEY TO DIAGRAM
Q through h² to axis, pivot through l to edge of grid and go horizontally
S through h² to axis, pivot through t to edge of grid and go vertically.
At intersection of horizontal and vertical lines read θm l/θo
BASED ON USE OF 1" O.D. TUBING
VERTICAL SPACING OF PIPES = 5'-0"
FINAL MEAN TEMPERATURE DIFFERENCE AT A GIVEN LENGTH FROM INLET IN DEGREES PER DEGREE INITIAL TEMPERATURE DIFFERENCE
l - LENGTH OF PIPE - FEET
AXIS
h² - DIFFUSION CONSTANT
- FT²/DAY
- FT²/HR
Q - FLOW - GALLONS PER MINUTE
t - TIME - DAYS
S - HORIZONTAL SPACING OF PIPES - FEET
FT²/DAY
FT²/HR
AXIS
h² - DIFFUSION CONSTANT

Fig. 9—Mean temperature of concrete at a given length from inlet

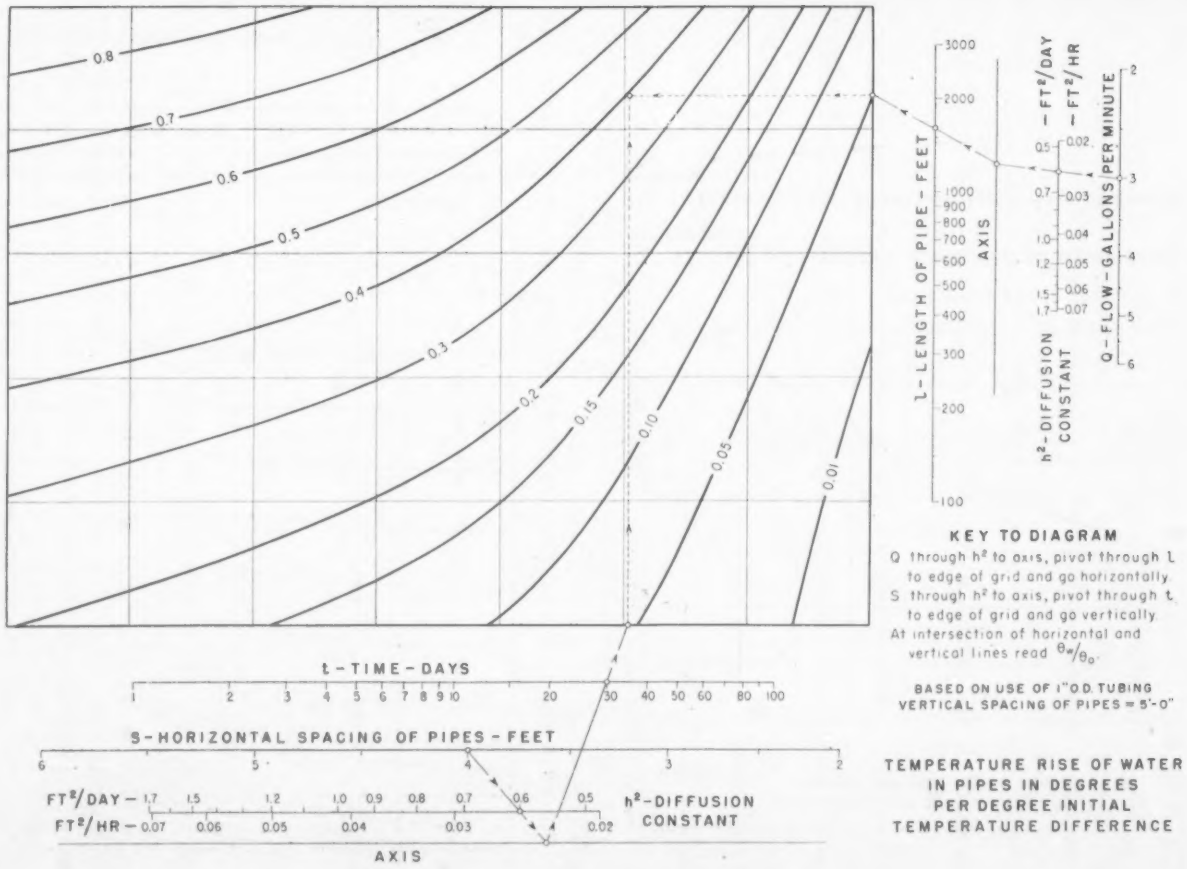
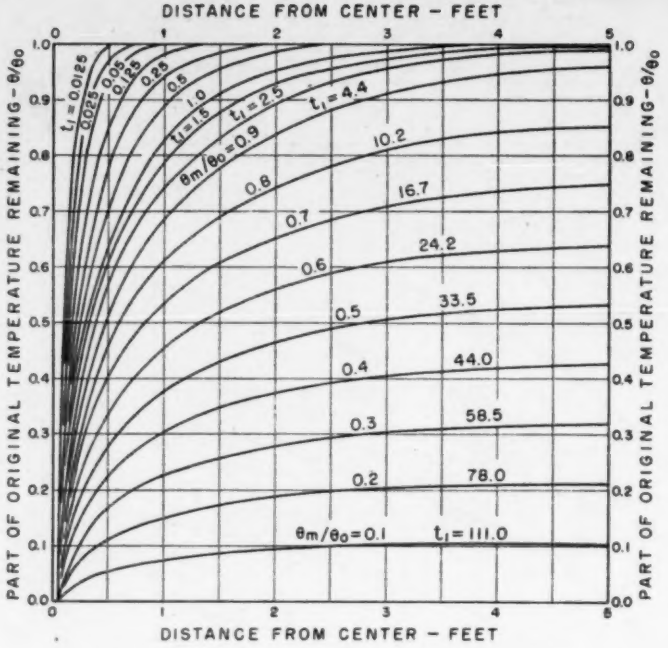

0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.15
0.10
0.05
0.01
t - TIME - DAYS
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 80 100
S - HORIZONTAL SPACING OF PIPES - FEET
6 5 4 3 2
FT²/DAY - 1.7 1.5 1.2 1.0 0.9 0.8 0.7 0.6 0.5
FT²/HR - 0.07 0.06 0.05 0.04 0.03 0.02
h² - DIFFUSION CONSTANT
AXIS
L - LENGTH OF PIPE - FEET
3000 2000 1000 900 800 700 600 500 400 300 200 100
AXIS
h² - DIFFUSION CONSTANT
Q - FT²/DAY
Q - FT²/HR
0.5 0.7 1.0 1.2 1.5 1.7
0.02 0.03 0.04 0.05 0.06 0.07
Q - FLOW - GALLONS PER MINUTE
2 3 4 5 6
KEY TO DIAGRAM
Q through h² to axis, pivot through L to edge of grid and go horizontally.
S through h² to axis, pivot through t to edge of grid and go vertically.
At intersection of horizontal and vertical lines read θ/θ₀.
BASED ON USE OF 1" O.D. TUBING
VERTICAL SPACING OF PIPES = 5'-0"
TEMPERATURE RISE OF WATER IN PIPES IN DEGREES PER DEGREE INITIAL TEMPERATURE DIFFERENCE

Fig. 10—Temperature rise of water in cooling coils



CONDITIONS

Uniform initial temperature distribution.
Outside surface insulated.
Inside surface kept at 0 temperature.
$\frac{b}{a}$ = 100, b = outer radius, a = inner radius.
Diffusion constant = $h_1^2 = 1.0 \frac{ft^2}{day}$
Diameter = D_1 = 10 Ft.

NOTATION

θ = Temperature at time given.
θ_o = Initial temperature.
θ_m = Mean temperature of cylinder.
t_1 = Cooling time – days.

FOR OTHER CONDITIONS, SAME $\frac{b}{a}$; $\frac{h_1^2 t_1}{D_1^2} = \frac{h_2^2 t_2}{D_2^2}$

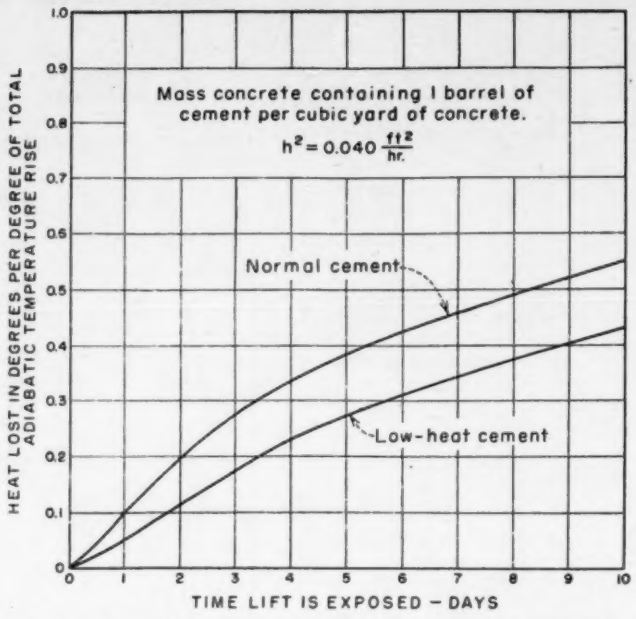

HEAT LOST IN DEGREES PER DEGREE OF TOTAL ADIABATIC TEMPERATURE RISE
Mass concrete containing I barrel of cement per cubic yard of concrete.
$h^2 = 0.040 \frac{ft^2}{hr.}$
Normal cement
Low-heat cement
TIME LIFT IS EXPOSED – DAYS

Fig. 11 (left)—Temperature distribution in concrete surrounding cooling pipe

Fig. 12 (above)—Loss of heat from top surface of five-foot lift of concrete

$$\frac{h_1^2\, t_1}{D_1^2} = \frac{h_2^2\, t_2}{D_2^2} \quad \ldots\ldots (10)$$

The subscript 1 refers to the conditions represented in the figure and the subscript 2 to the case to be calculated. From this relationship it is seen that the time required to cool to a given temperature varies as the square of the diameters of the cooled cylinders. It was thus more economical in cooling Boulder Dam to use a 5-ft. vertical spacing of 1-in. pipe rather than a 10-ft. spacing of 2-in. pipe.

i. Effect of size of cooling pipes—Increasing the size of the cooling pipe without increasing the spacing has the effect of decreasing the ratio of the outer radius to the inner radius $\frac{b}{a}$ of the cooled cylinder. It has been determined that the rate of cooling varies approximately inversely as the logarithm of the ratio for values of $\frac{b}{a}$ greater than 10. In other words to reduce the ratio from $\frac{b}{a} = 100$ to $\frac{b}{a} = 10$ suffices only to double the cooling rate. Since the cooling rate is proportional to the diffusion constant, the effect of altering the size of the cooling pipe may be calculated by substituting for the actual diffusion constant h_2^2 a fictitious constant h_f^2 obtained from the relationship

$$\frac{h_f^2}{h_2^2} = \frac{\log \left(\frac{b}{a}\right)_1}{\log \left(\frac{b}{a}\right)_2} \quad \ldots\ldots (11)$$

where $\left(\frac{b}{a}\right)_1$ is the ratio 100 for which the curves of Fig. 11 were prepared and $\left(\frac{b}{a}\right)_2$ represents the actual case.

j. Loss of heat from surface of lift—With a 5-ft. lift, a considerable proportion of the total heat developed by hydration of the cement may be lost from the top surface before the next lift is placed, as is indicated for concrete of average thermal characteristics by Fig. 12. Such curves, or the more complete curves of Fig. 41 of reference (13), may be used to obtain the average temperature rise of the lift without artificial cooling and without considering the effects of solar radiation, but give no indication of the temperature distribution through the lift. More favorable temperature distributions through the lifts were obtained in Friant Dam where the cooling pipes were spaced 2½ ft. apart horizontally on the rock surface and on the top of each 5-ft. lift. Thus during the early

period when the temperature normally rises, zoned regions having different temperatures developed through the lift; cool zones at the contact surface with the rock or the lift below and at the top where the lift was exposed, and a warm zone in the middle of the lift. Considering that very little compression is developed in concrete during the initial temperature rise, the state of zero stress through the lift was obtained with the center of the lift some 12 to 15 degrees higher in temperature than the top and the bottom of the lift. Subsequent reduction in temperature of the middle of the lift must have induced tensile stresses in this region, with balancing compressive stresses in the top of the lift and at the contact surface where cracks normally have their beginning. With this procedure, 5-ft. lifts 260 ft. long have been placed upon rock foundations where the restraint must have been close to 100 percent without having cracks develop in the concrete. The effects of thickness of lift and time of exposure of lift upon temperatures are also discussed by Savage[13], and elsewhere by McHenry[10] and Carlson[5]. Methods of computation of loss of heat from the surface for determination of average temperature distribution have been presented by Carlson[4], Savage[13], and Glover[8]. Schmidt's method may also be used to advantage in this problem. Use of the method presented by Savage requires that the shape of the temperature-rise curve be expressed by an exponential equation of the form

$$T = T_o (1 - e^{-mt}) \quad \ldots\ldots (12)$$

where T is the temperature rise in degrees at any time t in hours, T_o is the ultimate temperature rise, and m is a constant selected to make the mathematical curve fit the experimentally determined temperature-rise curve of the concrete. The variable mt is then employed to determine the percentage of total heat which has been lost at any time. The problem is thus complicated by the necessity of determining values of m for each type of cement. To have the mathematical curve fit the experimental curve over a long period of time usually requires an expression involving more than one term, such as

$$T = T_1 (1 - e^{-m_1 t}) + T_2 (1 - e^{-m_2 t}) \quad \ldots\ldots (13)$$

where

$$T_1 + T_2 = T_o.$$

Equations containing a single term which fit reasonably well the average curves of adiabatic temperature rise of Fig. 2 during the first 10 days are as follows:

Low-heat cement	$T = 39 (1 - e^{-0.025 t})$
Modified cement	$T = 53 (1 - e^{-0.030 t})$
Normal cement	$T = 63.5 (1 - e^{-0.035 t})$

Values of the coefficients of the brackets in these equations represent equivalent ultimate temperature rises which can be used only to compute the amount of heat lost during the first 10 days. Ordinarily the top of the lift is covered within 10 days, in which case the given values of m and T_o are applicable for determining the loss at any time within this period. The method developed by Carlson is simpler and more readily applied than either that of Savage or Glover. Schmidt's method is the simplest of all and is preferred in most cases. Objection to Carlson's and to Schmidt's methods is that the temperature at any given time cannot be determined directly but must be determined by a step-by-step process involving increments of the total time.

ACKNOWLEDGMENT

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(16) Vogt, Fredrik, "Analysis of Rise in Temperature During Hardening of Concrete," 1933. Obtainable from F. Bruns Bokhandel, Trondheim, Norway.

Discussion of this paper should reach the ACI Secretary, in triplicate, by April 1, 1945, for publication in the JOURNAL for June 1945.

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Discussion of a paper by Clarence Rawhouser:

Cracking and Temperature Control of Mass Concrete*

By. DUFF A. ABRAMS and AUTHOR

By DUFF A. ABRAMS†

The US Bureau of Reclamation has considered the control of cracking so important that it has dictated nearly all other features of the design and construction of large concrete dams. This problem has been before that organization for many years; scores of papers and reports have been issued on its various phases since 1930.

In accordance with the best estimate of the writer, the USBR has, during the past 12 years used its embedded-pipe system of cooling on eight large dams with a total volume of about 27,000,000 cu. yd. of concrete, and has expended on various phases of temperature control about the following:

Studies of special cements, thermal properties and cooling of concrete	$ 500,000
Extra cost of special cements, 27,000,000 bbl. at 10¢	2,700,000
Cooling 27,000,000 cu. yd. at 23¢	6,200,000
Instruments embedded in concrete	250,000
Reading instruments, calculations and reports	200,000
Unclassified	150,000
Total	$10,000,000

The foregoing are the direct costs of crack and temperature control, by the use of special cements and cooling of concrete. If we include the cost of vertical contraction joints and grouting, the total would be about $20,000,000.

SUMMARY AND CONCLUSIONS

The paper is the end product of 15 years of continuous attention to "cracking and temperature control of mass concrete," by the US Bureau of Reclamation. About 39 of the 42 pages were devoted to "temperature

*ACI Journal, February 1945; *Proceedings* V. 41, p. 305.
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control," but we search these pages in vain for any evidence that temperatures in large concrete dams *either have been or can be controlled.*

In the writer's opinion, the method of control recommended by the author contains a number of fallacies that make it wholly ineffective. The principal fallacy is the assumption that portland cement ceases to hydrate when artificial cooling is discontinued. The operation of these fallacies will be demonstrated by specific reference to the results obtained in cooling of Boulder Dam, where the low-heat-cement-embedded-pipe system, recommended by the paper was used.

Data and arguments presented below will show:

1) That the author withheld the only type of information of importance, namely, existing temperatures in large concrete dams,

2) That every important assumption on which the USBR based its theory and practice of cooling Boulder Dam was erroneous,

3) That the isotherms published since completion of Boulder Dam, as representing interior temperatures, are misleading,

4) That the low-heat-cement-embedded-pipe system of temperature control was ineffective at Boulder Dam, and accomplished neither what it was designed for nor what has since been claimed,

5) That hydration of cement in Boulder Dam did not stop when the refrigerator was turned off. Cooling of the concrete was only an unimportant interlude in the hydration of this cement. About 50 per cent of the total heat of hydration is still in the dam,

6) That Boulder Dam was not cooled to its "final stable temperature." The writer estimates the temperature in the lower part of the dam to be about 64 F. higher than that assumed in design,

7) At points indicated in official Boulder Dam charts by the 56 F. isotherm, the temperature is now of the order of 120 F.,

8) Present temperature is probably higher than the average maximum reached by the concrete. All benefits of cooling Boulder Dam have been lost by the end of 12 years,

9) Due to unconsidered temperature rise, the computed concrete stress in Boulder Dam is 2800 psi. above that considered in design,

10) The writer's estimate of temperature rise in Boulder Dam is confirmed by observations of men who were on the job during construction or shortly after completion,

11) That "low-heat" portland cement is a misnomer,

12) That a large, rapidly-built dam, using 1 bbl. per cu. yd. of low-heat cement, cannot be adequately cooled by a system applied for "a month or two during the period of construction,"

13) That grouting of Boulder Dam was a mistake,

14) Experience at Boulder Dam contradicts the principal recommendations of the paper.

The paper does not offer a solution to the problems of cooling a large, rapidly-built concrete dam. These problems cannot be solved by ignoring them. Over 400 thermometers were embedded in Boulder Dam; the author is requested to give the temperatures observed in the section in Fig. A, at 2, 3, 6, and 12 years after concreting began.

TEMPERATURE CONTROL IN CONCRETE DAMS

After setting forth the merits of special portland cements and the mechanics of the embedded-pipe system of cooling and grouting of concrete dams, the author sums up as follows (p. 322):

The concrete may be *cooled to the final stable temperature in a month or two during the period of construction* after which, with the concrete at minimum volume and the contraction joints opened to the fullest extent, *the joints may be grouted*, thus obtaining the monolithic structure assumed in design.

Certain words are italicized to direct attention to them.

The writer shows that, as a consequence of the properties of portland cements, and by specific reference to Boulder Dam:

a) That under these conditions the concrete cannot be "cooled to the final stable temperature in a month or two during the period of construction,"

b) That the embedded-pipe system of cooling used at Boulder Dam did not accomplish what was claimed for it, although, in some instances, cooling continued for 6 months after concreting,

c) That, although grouting was delayed for more than a year after the concrete was placed in the lower part of the dam, the grouting of Boulder Dam was a mistake.

OFFICIAL INFORMATION ON BOULDER DAM COOLING MISLEADING

The foregoing quotation from the paper is a statement in somewhat different words of what has been claimed for the low-heat cement and the same system of cooling as used on Boulder Dam. A number of reports by USBR writers which give the engineering profession "official" information concerning temperatures in Boulder Dam will be cited. These reports were published 1 to 6 years after the dam was completed.

1936. A year after the completion of the dam, J. L. Savage, Chief Designing Engineer, published a book, "Special Cements for Mass Concrete." In discussing the cooling of Boulder Dam, Mr. Savage gave a chart, bearing the title, "Section of Boulder Dam Showing Isotherms and Location of Embedded Instruments." This chart (Fig. 54) is reproduced here as Fig. A.

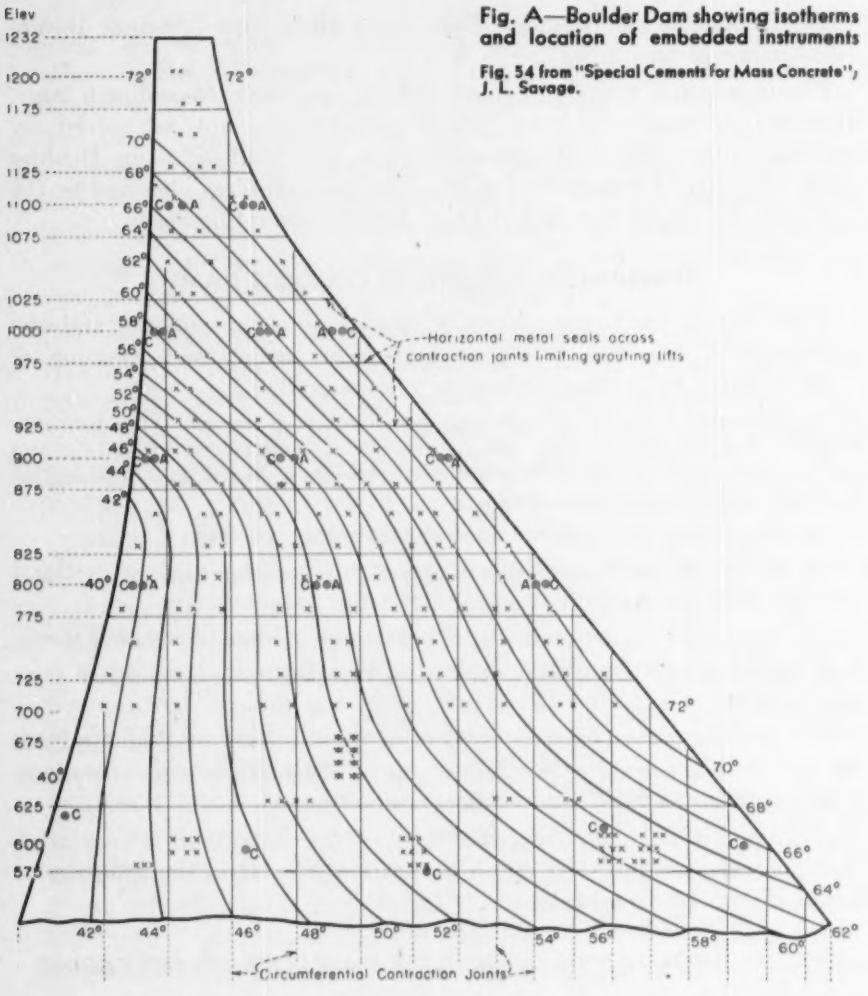


Fig. A—Boulder Dam showing isotherms and location of embedded instruments

Fig. 54 from "Special Cements for Mass Concrete", J. L. Savage.

× Location of thermometers with respect to faces of dam

⊕ Location of sets of 10 strainmeters, A at abutments, C in center of dam

Isotherms indicate temperatures to which concrete was cooled.

1938. In his paper "Temperature Control of Mass Concrete in Large Dams," in "Dams and Control Works," a book published by USBR 3 years after the completion of Boulder Dam, Mr. Rawhouser gave a chart which is essentially the same as that by Mr. Savage.

1941. A book "Boulder Dam" (Boulder Dam Project Final Reports, Bulletin IV-2) p. 190, published 6 years after the completion of the dam used the same chart as Savage and Rawhouser.

In addition to the isothermal charts mentioned above, many USBR reports published since the completion of Boulder Dam, contain direct statements of the results of the cooling operations. The following are typical:

1936. J. L. Savage in "Special Cements for Mass Concrete", (Cooling of Boulder Dam) p. 146:

The great proportions and rapid rate of construction made it imperative that some means be taken to remove from the concrete the heat developed during cement hydration, and that existing initially in the concrete above the *final temperatures* desired in the dam.

1940. "Thermal Properties of Concrete", (Boulder Dam Project Final Reports, Bulletin VII-1) p. 9:

In the construction of an arched gravity dam it is important that *cooling and complete contraction of the concrete,* followed by grouting of the contraction joints between the columnar blocks, be accomplished during the construction period.

1941. "Boulder Dam", (Boulder Dam Project Final Reports, Bulletin IV-2) p. 33:

Therefore, the problem became one of devising a construction program which would result in the attainment of *final stable temperatures,* and consequently final stable volume, during the regular period of construction. A program of artificial cooling of mass concrete together with the use of low-heat cement was adopted as the most feasible method of solving the problem.

From the above charts and frequently-reiterated statements of the benefits of low-heat cement, cooling, and that "final stable temperatures" had been reached, the reader might readily gain the impression that such material represented actual temperatures as determined by the 416 thermometers that were embedded in the dam. But that would be a grave mistake; the charts gave nothing of the kind. The "isotherms" are purely theoretical curves—made up in Denver several years before concreting on the dam was begun. Any of the above 4 reports might have given actual concrete temperatures that would have been of utmost interest and value to engineers, but all gave, instead, the original theoretical isotherms, or merely repeated the aims of the designers of the dam.

It will be shown that the cooling system used at Boulder Dam did not produce "final stable temperatures", and that the temperatures after completion of the dam were vastly different from those indicated by "official" statements.

HYDRATION OF LOW-HEAT CEMENTS

As a background for our consideration of temperature control, it is necessary to present a brief discussion of the hydration of portland cements. The discussion will be confined largely to low-heat cements;

this type was recommended by the author for use in conjunction with artificial cooling.

The only value portland cement has as a structural material is its property of "hydration", that is, the formation by chemical reactions with water of new and stable compounds. There are a number of different manifestations of cement hydration, all of which have significance in mass concrete:

a) Strength of concrete,
b) Temperature rise in concrete,
c) Heat of hydration of cement,
d) Quantity of water fixed by hydration.

It has been known for nearly 100 years that the strength of hydraulic cement increases with age. The writer was probably the first to demonstrate that, so long as concrete remains in a damp condition and at a favorable temperature, the compressive strength continues to increase essentially as a direct function of the logarithm of the age. Many series of tests are now available that show this increase to continue indefinitely. In one series tests have been made up to 30 years. There is no reason to suppose this increase will not continue indefinitely according to the logarithmic law.

Tests of temperature rise have shown results exactly parallel with those of compressive strength. If curves in Fig. 1 of the paper were plotted to a logarithmic scale of ages, they would give essentially straight lines at ages of 3 to 28 days. Similar tests of low-heat cements reported in other USBR publications show this logarithmic relationship to hold up to one year. Some of these tests are referred to below.

Similar results have been found for heat in hydration; in fact temperature rise of concrete, in the absence of loss or gain of heat from outside sources, furnishes a direct means of computing heat of hydration.

A paper on "Water Retained by Hardened Cement Pastes", by Raymond Wilson and F. A. Martin (ACI JOURNAL Jan.-Feb., 1935 Proceedings V. 31, p. 272) showed that at 3 days to 1 year the water retained by hardened pastes, increased as a logarithmic function of age.

Of the above measures of the hydration of portland cements the compressive strength has the longest history—30 years. Since all of the factors listed above are different manifestations of the same reactions, it is a safe assumption that all of them will show a continuous increase without known time limit, and that this increase will closely follow the logarithmic law.

Fig. B gives 4 curves, all based on USBR tests:

Curve A. Heat of hydration of Boulder Dam low-heat cement; tests reported in "Special Cements for Mass Concrete", by J. L. Savage, 1936, p. 152.

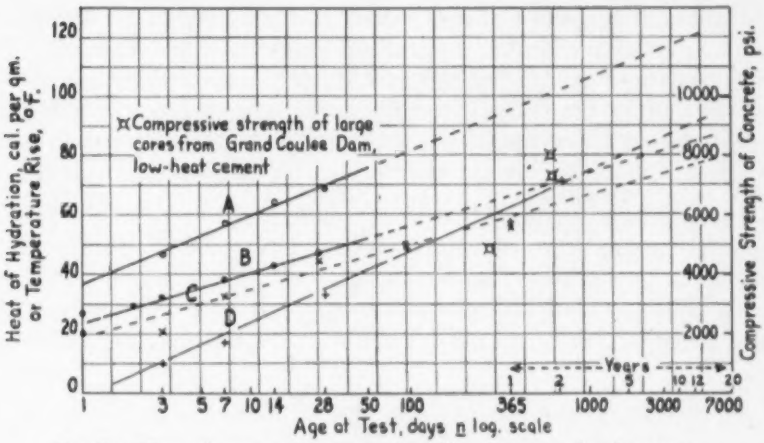

Heat of Hydration, cal. per gm. or Temperature Rise, °F.
Compressive Strength of Concrete, psi.
Compressive strength of large cores from Grand Coulee Dam, low-heat cement
A
B
C
D
Years
Age at Test, days n log. scale

Fig. B—Effect of age on the properties of low-heat portland cements.

Curve A. Heat of hydration, Boulder Dam cement.
Curve B. Temperature rise, low-heat cement in paper.
Curve C. Compressive strength of concrete, 10 standard cements.
Curve D. Compressive strength of concrete, 13 low-heat cements.

Tests by US Bureau of Reclamation. See text for complete references.

Curve B. Temperature rise of low-heat cement, from Fig. 1 of the paper.

Curve C. Compressive strength of concrete; average of 10 *standard* portland cements; from USBR "Concrete Manual", 1942, p. 451; cured in fog room at 70F.

Curve D. Compressive strength of concrete; average of 13 *low-heat* cements; parallel to tests in Curve C. The concrete strengths at ages of 3 days to 1 year age given below—also parallel strengths of *modified* cement.

All of the curves in Fig. B are represented by expressions of the form:

$$v = a + b \log n$$

where v is the vertical ordinate, a and b are constants; n is age at test in days. In the figure a is the 1-day ordinate, and b is the slope of the curve (to log. scale). b is equivalent to the ordinate at 10 days minus that at 1 day. This makes the derivation of the constants quite simple; for example Curve B, temperature rise of low-heat cements, has the expression: $t = 24 + 16 \log n$. The indicated value of 1 day is a little higher than that determined by test, but the expression gives almost exact values for later ages.

When a variable increases as a logarithmic function of age, it means that the rate of increase becomes smaller and smaller. For concrete strength, the increase between 3 and 12 years would be the same as that between 3 and 12 months, or 3 and 12 days. Similar rates apply to heat of hydration and temperature rise.

The near-parallelism of the curves for heat of hydration, temperature rise, and concrete strength is accidental and results from the particular vertical units and scales used, but it emphasizes the similarity of

three different measures of hydration. Concrete strength may be used as a measure of the heat of hydration. Curves A and D show that for this cement and mix, after the first day, each increment of 1000 psi. was the result of the evolution of 10 cal. per gm. of heat. Further study would, no doubt, establish a general low of this type for other cements and mixes.

The important feature of these curves for purposes of this discussion is that all of these properties of the low-heat cements continue to increase as a logarithmic function of age, within the limits of the tests. We are justified in the belief that these properties will continue to increase with age, as indicated by the curves projected beyond the test limits.

Data in USBR "Concrete Manual", 1942, p. 451, may be summarized, using strengths scaled from curves:

Type of Portland Cement	Number of Samples	Compressive Strength of Concrete					Ratio 1 yr. to 28 d.
		3 d.	7 d.	28 d.	90 d.	1 yr.	
Standard.....	10	2100	3300	4500	5000	5700	1.27
Modified.....	10	1700	2500	4100	5100	5500	1.34
Low-Heat.....	13	1000	1700	3300	4900	5800	1.76

Water ratio 0.54 to 0.56, by weight; cured at 70F. in fog room.

The tests show:

a) The low-heat cements had low early strength, but at the end of 1 year gave concrete strength higher than either the standard or modified,

b) From 28 days to 1 year, the rate of increase in strength was much higher for the low-heat than for the other types of portland cement.

There is no reason to believe the low-heat cement would not continue to gain in strength at the higher rate up to 12 years, as shown by Curve D. This, of course, is only another way of saying that heat of hydration will also develop at a higher rate after the first year.

The first concrete placed in Boulder Dam is 12 years old; since there is no known method of stopping the hydration we are justified in concluding that the total heat of hydration of this cement up to date (Curve A) is about 120 cal. per gm., and that the low-heat cements in Curve D, would at 12 years, give a compressive strength of about 9000 psi. The points plotted for compressive strength of cores cut from Grand Coulee Dam (low-heat) at ages of 9 to 21 months, give a measure of the concrete strengths to be expected at later ages.

When measured in terms of 5 to 12 years, the low-heat portland cements probably have a higher heat of hydration than the standard portlands in use at the time Boulder Dam was under consideration. These low-heat cements should have been designated "low-early strength."

The most casual and elementary consideration of hydration of portland cement demonstrates that the hydration of the cement in Boulder Dam did not cease when the refrigerator was turned off after cooling the concrete "a month or two".

The most certain method of restricting temperature rise in a large concrete dam is to reduce the heat-producing element to a minimum. It is working at cross-purposes to put in heat-producing material that is obviously not needed, then go to great expense in removing a portion of the unwanted heat. When we buy portland cement, we are in reality paying for Btu., and no more should be bought than can be used to advantage.

The above considerations raise anew the whole question of special cements for mass concrete. The USBR solved this problem in the construction of Elephant Butte Dam, where a sand-cement was used, consisting of equal parts of portland cement and pulverized sandstone, interground to exceed the fineness of cement. This dam is 306 ft. high; when built it was one of the highest masonry dams in the world. The method saved about ½ the normal cost of cement, and required no expenditures for cooling or grouting. This dam is 30 years old; it is probably as free from cracks today as any large concrete dam in existence.

ESTIMATED TEMPERATURES IN BOULDER DAM

Concreting on Boulder Dam was begun June 1933; by the end of 1934 the bulk of mass concrete was in place. The first concrete is now 12 years old. Scraps of information scattered through many USBR reports published during the past 11 years enable us to piece together a balance for the heat in this dam:

$$H = A + B + D,$$

where H is total heat of hydration of the cement,

A is heat lost during construction,

B is heat of hydration removed by artificial cooling,

C is heat used in depressing temperature of concrete below that at which it was placed,

D is heat still in the dam.

Numerical values of the symbols will be developed. It will become apparent below why C did not appear in this balance.

H. Total heat of hydration.

Fig. B shows that at 12 years Boulder Dam cement would be expected to have a heat of hydration of 120 cal. per gm. This permits an estimate of the total heat generated by hydration. 1 lb. is 454 gm.; 1 cal. is equivalent to 0.004 Btu. 1 lb. of the cement develops 454 x 120 x 0.004 = 218 Btu. This heat would raise the temperature of 26 gal. of water 1 F.

Each cubic yard of concrete contained 384 lb. of cement, and weighed 4220 lb. Cooling was applied to 3,290,000 cu. yd. The total heat generated by the cement would then be:

$H = 3{,}290{,}000 \times 384 \times 218 = 275{,}000{,}000{,}000$ Btu.

A. Heat lost in construction.

B. W. Steele in an article, "Cooling Boulder Dam Concrete", *Eng. News-Rec.* Oct. 11, 1934, stated that the heat lost in the interval between the placing of 5-ft. lifts, which averaged 4.7 days, was roughly 1/3 of the total heat generated. The significance of his statement is not altogether clear, but he obviously referred to the amount of heat generated at 4.7 days, since there was no appreciable loss after the blocks were covered by the next lift. Fig. B shows heat of hydration of Boulder Dam cement at 4.7 days to be 51 cal. per gm. The loss would then be 17 cal., or 14 per cent of the 12-year value of 120 cal. per gm., so that total heat lost was

A = 39,000,000,000 Btu.

B. Heat of hydration removed by cooling.

USBR Bulletin IV-2, "Boulder Dam Project Final Reports, Design and Construction of Boulder Dam", 1941, p. 199, stated: "The total amount of heat extracted from the concrete was about 159 billion Btu. . . . . "

But we must go to still other USBR papers before we obtain information necessary for setting up a balance sheet for the heat of hydration in Boulder Dam. The book, "Special Cements for Mass Concrete", by J. L. Savage, 1936, p. 2, gave curves, "Average Placing, Maximum and Final Temperatures in Boulder Dam". In using these curves, it was necessary to make allowances for 2 factors: a) Temperatures for the first month or two were omitted; b) Final temperatures for the last few months were probably not representative of the large bulk of the concrete placed, on account of reduced volume placed as the top of dam was approached.

The curve "Observed avg. max. temp. of concrete placed at time plotted", shows an average of about 108F. Similarly, from the curve, "Observed initial temp of concrete", we find about 72F; and from the curve, "avg. final temp of concrete" we find 55F. The same curves were given to a reduced scale in a paper, "Cracking of Mass Concrete",

by Blanks, Meissner and Rawhouser, ACI JOURNAL, Mar.-Apr., 1938, *Proceedings*, V. 34, p. 484.

The drop in temperature, as a result of cooling was then, 108 − 55 = 53F. We know the quantity of concrete cooled and its unit weight. Specific heat, 0.215. With this information we can compute the amount of heat used in dropping the temperature 53F:

3,290,000 x 4220 x 0.215 x 53 = 159,000,000,000 Btu.

This agrees with USBR value for the total heat removed by cooling.

It is important to note that the above value includes both *B* and *C*. We must now segregate these quantities. The heat used in removing heat of hydration was proportional to the drop from maximum to placing temperature, or 36/53 of heat removed by cooling; hence:

$$B = \frac{36 \times 159{,}000{,}000{,}000}{53} = 108{,}000{,}000{,}000 \text{ Btu.}$$

C. Heat used in depressing temperature of concrete.

In a similar manner we find:

C = 51,000,000,000 Btu.

Heat balance.

With the information at hand, we can set up a balance sheet for the heat in the Dam (units of 1,000,000,000 Btu):

H = Total heat of hydration	275
A = Heat lost in construction	39
B = Heat of hydration removed in cooling	108
C = Heat used in depressing temp. of concrete	51
Total heat of hydration removed (A + B)	147
D = Heat still in dam *H* − (A + B)	128

D. Heat still in dam.

It is important to note that we are concerned here only with the *heat of hydration removed* by cooling, hence *C* does not enter into the final computation. *C* consumed 51/159 = 32 per cent of the heat removed by cooling, but it *did not remove any heat of hydration*. To illustrate this point further, the temperature of a block of steel might be depressed 50 F. We can compute the quantity of heat extracted, but it is obvious that no heat of hydration was removed.

The above figures show that 53 per cent of the total heat of hydration was lost in construction, or was removed in cooling, and that 47 per cent, or 128,000,000,000 Btu. is still in the dam.

Temperature rise.

All USBR writers agree that no heat was lost after cooling, hence

the 128,000,000,000 Btu. still in Boulder Dam can have but one effect, that is, to raise the temperature of the concrete:

$$\frac{128{,}000{,}000{,}000}{3{,}290{,}000 \times 4220 \times 0.215} = 43 \text{ F.}$$

Curve B in our Fig. B shows the low-heat cements of the paper would have a temperature rise of 85F. at 12 years. Since 47 per cent of the heat remains in the dam we would expect, on this basis, a rise of 0.47 x 85 = 40F., as the rise computed from independent data. This is essentially the same as the 43F. computed above based on the indicated heat of hydration of Boulder Dam cement.

Heat from other sources.

There are 2 other factors which should be considered, each of which would cause a heat gain, in addition to the *normal* hydration of the cement considered above: *a*) Heat absorbed from foundation rock, b) Hydration of cement at an increasing rate due to temperature rise.

Heat absorbed from foundation rock.

In the design of Boulder Dam it was assumed that the foundation rock was permanently at the mean annual temperature of 72F, and that the concrete in contact with rock could be cooled to, and would remain at, its "final stable temperature". These assumptions involved a number of fallacies, 3 of which will be mentioned:

72F was too low for the mean annual temperature,

The foundation rock is permanently at about 100F. This is about mean *sun* temperature.

It is obvious that concrete cannot remain at a temperature of 40 to 60F when it is in contact with an unlimited volume of rock at 100F.

It seems a safe assumption that the temperature of the lower part of the dam was quickly raised by contact with the warm rock. It is impracticable to be exact, but the writer estimates an increase in temperature up to this time near the bottom of the dam of 11 F. due to this cause.

More rapid hydration of cement.

It is a practically invariable principle of chemistry that the rate of reaction increases with a rise of temperature. Since the lower part of Boulder Dam would soon be heated by absorption from the rock, it is doubtful whether the rate of hydration of the cement was seriously retarded as a result of the cooling operations. On the other hand, the rate would almost certainly be increased as the concrete became heated. It appears to be a conservative estimate that the over-all effect of this increased rate of hydration over 12 years, would raise concrete tem-

perature about 10 F, in addition to that already computed as due to the normal rate of hydration.

Present temperature of Boulder Dam.

We can now make a synopsis of estimated temperature in the lower portion of the dam:

Average temperature after cooling	56F
Rise due to normal hydration of cement	43
Rise due to warm rock	11
Rise due to hydration at a higher than normal rate	10
Estimated present temperature	120F

It is the writer's opinion that the temperature along the lower portion of the central isotherm in Boulder Dam, marked "56F", in Fig. A is now of the order of 120F, or about 64 F. higher than that assumed in the design.

TEMPERATURE STRESS IN BOULDER DAM

In their efforts to avoid cracks in Boulder Dam, USBR engineers seem to have created another problem that may be even more difficult to solve. If we assume a coefficient of thermal expansion of concrete of 0.0000055, and a modulus of elasticity of 8,000,000 psi., a temperature rise of 64F in restrained concrete represents a compressive stress of:

$0.0000055 \times 8{,}000{,}000 \times 64 = 2800$ psi.

The actual stress represented by this figure is problematical, since we do not know how to evaluate the combined effects of time, temperature, multi-axial stress, etc. At best a computed stress of 2800 psi., over and above that for which the dam was designed, suggests the necessity for a candid reconsideration of the stress to which Boulder Dam is now subjected.

The assumed modulus may appear too high, but the writer considers it conservative for 12-year old concrete, in view of actual tests on 18- and 36-in. cylinders which gave 5,200,000, at 28 days, as reported by J. L. Savage, "Special Cements for Mass Concrete", 1936, and over 6,000,000 at 1 year, as reported by Blanks and McNamara, ACI JOURNAL Jan.-Feb., 1935.

BOULDER DAM COOLING CONTRADICTS PAPER

The foregoing figures on the cooling of Boulder Dam contradict many of the most significant statements of the paper. It is plain that the cooling system used at Boulder Dam accomplished neither what it was designed for nor what has since been claimed for it. Our knowledge of the hydration of cement makes it apparent that no system of artificial cooling of concrete can adequately remove the heat of hydration from a

large, rapidly-built dam in "a month or two during the period of construction", as stated in the paper. The bulk of the heat of hydration cannot be removed before it is generated. The combined effects of dropping temperature of concrete and "subcooling" are generally limited to a drop of 15 to 20 F. below placing temperature; hence only an insignificant quantity of heat can be accounted for in this way.

The fact that the writer's figure of 159,000,000,000 Btu. removed from the dam agrees with the total reported by USBR, indicates that cooling produced no appreciable effect before the concrete reached its maximum temperature. This is a reasonable conclusion, since cooling circuits included several adjacent blocks, and no block could be cooled until the last block in the group was concreted. This raises a question as to the applicability of the curves in Fig. 1 of the paper, marked "Average temperature rise, 5 foot lifts exposed to air". It is not apparent how cooling can become effective in 1 or 2 days, as indicated by these curves, nor is it clear how artificial cooling can be "started immediately after placing the concrete", as stated (p. 322) unless each block has a separate cooling circuit, which is generally impracticable and was not the practice of the USBR on such dams as Boulder, Grand Coulee, Shasta.

CORROBORATING TESTIMONY OF USBR WRITERS

The estimate of present temperature in Boulder Dam may at first appear to be "theoretical". It was of course, based essentially on 3 thoughts: a) Known quantities of heat lost or removed by cooling; b) Known properties of the concrete; c) The belief that there is no way to prevent the continued hydration of cement in a large dam.

But it is interesting to note that various USBR writers have, during the past 11 years, given testimony that fully corroborates the writer's conclusions. Four such instances will be cited:

1934. B. W. Steele, in the article referred to above, written about September 1934, when two-thirds of the concrete was in place, showed actual temperatures at 49 points in Boulder Dam. At 2 points near the bottom, in concrete placed during the previous summer, he recorded temperatures 8 and 13 F. higher than those desired. At these points the concrete was supposedly "subcooled" 5 to 10 F. below the isotherms shown in Fig. A; and cooling had been discontinued only 6 months. In other words, temperature here was 13 to 23 F. higher than intended.

1934. In a report, "Movement of Boulder Dam Due to Grouting Contraction Joints", (USBR Technical Memorandum 496) by A. W. Simonds, we find a statement on this subject, describing the conditions observed before grouting was begun in May, 1934, and when the lower concrete was less than 1 year old:

At this time it was discovered that the dam was warming up more rapidly than desired after cooling had been terminated in the completed section.

1936. In a report, "Rock Temperatures in Black Canyon", (Tech. Memo. 530) by J. G. Ross, we find:

Resistance thermometer number 77 embedded in block L-7 at elevation of 580 is approximately midway between upstream and downstream faces and less than 35 feet from the Nevada abutment. The concrete in this vicinity, after being poured, was artificially cooled to 49 degrees in December, 1933 after which cooling was discontinued. Within one year the temperature in this block gradually rose to 70 degrees and, by the end of the second year after artificial cooling (December, 1935), the temperature had risen to 80 degrees. Other thermometers in the same block verify these figures. Temperatures in nearby blocks show the same tendencies although in none of them were temperatures higher than 77 degrees recorded.

In Fig. A the location of thermometer 77 can be identified from the above description; it is in the middle cluster near the bottom of the dam, where the desired temperature was 53F.

Mr. Ross attributed this temperature rise to heat from foundation rock and to numerous hot springs that discharged large quantities of water of 100 to 114F. This explains part of the observed rise of 28 to 31 F. above the "subcooled" temperature, but it was no doubt partly due to hydration of the cement.

1944. The paper under discussion gave some evidence. It stated that at "one point" on each of the exposed faces of Boulder Dam the temperature is above that intended: Upstream face, midwinter, 1 F.; midsummer, 11 F. Downstream face, midwinter, 4 F.; midsummer, 16 F.

Summer temperatures would apply to about 2/3 of the year. While there are questions as to what was meant by "face" temperatures, and the location of "one point" on a face 900 ft. long, the figures show a rise of about 10 F. above that intended.

Summary of USBR evidence.

The foregoing examples cited from four different USBR reports over the past 11 years show:

a) That the temperature rise in Boulder Dam is real, and that the writer's estimate is not mere "theory",

b) Temperature rise was noted by men on the job when the first concrete was less than 1 year old and before grouting of vertical contraction joints was begun,

c) A few months after completion temperatures near the bottom of the dam were 15 to 31 F. above those intended,

d) Temperatures at the faces of the dam 9 years after completion were about 10 F. higher than intended,

e) That "final stable temperature" was not reached in "a month or two during the period of construction",

f) Actual temperature rise occurred at a more rapid rate than that suggested by the writer's analysis.

Practically all the important recommendations of the paper were contradicted by experience at Boulder Dam.

FURTHER COOLING OF BOULDER DAM IMPOSSIBLE

That those in charge of the design and construction of Boulder Dam overlooked the continuing hydration of the cement, is clearly shown by the fact that all connections with the cooling system of 590 miles of embedded pipes, were buried in the dam *and the pipes filled with grout*, soon after refrigeration was discontinued. This precluded any resumption of cooling at a later date.

GROUTING OF BOULDER DAM

The paper lists grouting of vertical contraction joints as: ". . . a special requirement of mass concrete in order to insure its integration and action as a continuous mass after it has been brought to the desired temperature."

USBR writers are responsible for a number of different estimates for the average widths of the contraction joints after cooling Boulder Dam. The most recent, and probably the most detailed statement is in Bulletin IV-1, "General Features", 1941, p. 157: "The average width of the joint openings was 0.126 in. for the radial joints, and 0.087 in. for the circumferential joints". The temperature rise required to close these joints would be:

Joint	*Average Size of Blocks*	*Average Opening*	*Temperature Rise to Close*
Radial	45 ft.	0.126 in.	42 F.
Circumferential	37	0.087	35 F.

The temperature rise necessary to close these joints is essentially that computed by the writer, when the cement was assumed to hydrate at its normal rate; and considerably less than the estimated temperature rise due to all causes. If these joints had been left open:

a) Most of the expansion of the concrete, due to unconsidered temperature rise, would have been taken care of,

b) The joints would have closed before the reservoir was full,

c) The excessive concrete stress computed above would have been avoided.

Grouting was not necessary to prevent leakage through the dam; this would have been prevented by the grout stops. These considerations make it appear that the grouting of Boulder Dam was a mistake, and that all benefits of cooling have been lost at the end of 12 years.

WANTED—BOULDER DAM TEMPERATURES

The only statement in the paper on temperatures in large concrete dams was that quoted above on "face" temperatures at 2 points on Boulder Dam. More than 400 thermometers were embedded in this dam, and thousands have been embedded in other concrete dams built by the USBR. These and thousands of other instruments give numerical data on "Cracking and temperature control of mass concrete", but this is one of the best guarded secrets in the world today. Not one reading of these instruments has been "officially" released during the past 11 years.

Since the present paper again brings the subject of temperature control to the fore, it seems to the writer that it is incumbent on the author to give the temperatures observed in Boulder Dam, at the section shown in Fig. A, at, say, 2, 3, 6, and 12 years after concreting began in June 1933.

AUTHOR'S CLOSURE

Temperature control of mass concrete was first considered of sufficient importance to warrant special measures when plans were formulated for the construction of Boulder Dam, which was nearly twice the height of any previously constructed concrete dam. The measures successfully used at Boulder Dam for the control of cracking have been the subject of study by designers and builders of dams over the entire world. It is only natural that as more information becomes available more exact and effective measures will be developed for coping with the troublesome problem of cracking of large concrete dams.

The paper presented a discussion of the methods of temperature control that are available and the effects of various measures that might be used. The subject matter was assembled primarily for use by ACI Committee 207, "Properties of Mass Concrete" in conjunction with other material assembled by the committee. It is appreciated that in presenting this information separately to others interested in the problems, the lack of specific data in the form of observed temperatures of actual structures weakens the presentation. Mr. Abrams calls attention to this lack of observational data. The paper as presented was not intended to be a report of conditions at Boulder Dam or any other specific structure. However, in reply to Mr. Abram's discussion, which focuses attention almost entirely on what has or has not taken place in Boulder Dam, some temperature data* are presented which, it is hoped, will correct the impression that these data are being purposely withheld, and will

*A more complete record of the temperature history of Boulder Dam is to be contained in one of the final reports on the Boulder Canyon Project, a bulletin entitled, "Cooling of Concrete Dams" now in preparation.

dispel any further desire to make wild conjecture as to the temperatures that actually exist.

No attempt will be made to answer point by point the 14 numbered conclusions and the additional lettered statements of conclusion throughout Mr. Abrams' discussion since many of these points are supported only by argument which presupposes that the present temperature in the interior of Boulder Dam is about 120 F. All that one needs to do to refute most of the argument is to look at the structure itself. The highest temperature recorded at present by any of the many thermometers embedded in the dam is 84 F. The temperature in the interior of the dam farthest removed from any surface is actually 75 F and has not increased perceptibly for more than a year.

Mr. Abrams' discussion of the hydration of low-heat cements leads him to the conclusion that the increase in adiabatic temperature rise will continue indefinitely in proportion to the logarithm of the age. Predictions to 12 years of age are made on this basis from observed data from age 1 day to 28 days. His discussion is based on the assumption that generation of heat exactly parallels development of strength and some short-time data for temperature rise are presented in support of this assumption.

Obviously, the logarithmic relationship of heat generation and age, or of strength and age, is not a rational relationship because it implies an ultimate heat generation which is unlimited in amount. It is also drastically in error in the interval between the time of casting and age one day. It is an empirical relationship that at best can be made to fit the data over a relatively short time and cannot be used for extrapolation of the curves or prediction of temperatures or strength far beyond the limits of observation. Study of a considerable amount of published and unpublished data of long-time strength development of concrete reveals that usually the data are fitted better by a curved line than by a straight line on the logarithmic time scale. This is true especially of large specimens or of cores extracted from service structures. Usually it is only with small specimens in the presence of excess moisture that a straight line will fit the strength data for more than a few years. Even with the short-time data presented in Fig. B of Mr. Abrams' discussion, the points of curves A and C can be fitted more closely by a curved line than by a straight line. Departure from the straight line is more apparent as time increases. Thus the basis of Mr. Abrams' discussion of hydration of low-heat cements is not supported by observed data. His subsequent computations of estimated temperatures and of temperature stress in Boulder Dam are, therefore, grossly in error.

Mr. Abrams states that the principal fallacy in the paper is the assumption that portland cement ceases to hydrate when artificial cooling is

discontinued. It is regretted that this meaning was read into the statements of the advantages of cooling the concrete artifically. Such meaning was not intended. At the time of the construction of Boulder Dam very few data were available on the long-time heat development of low-heat cement and it was estimated that under adiabatic conditions the temperature of Boulder Dam concrete would ultimately rise about 9 degrees above the value attained at age 28 days. Statement of this expectation was made in the Bureau of Reclamation publication, "Special Cements for Mass Concrete", by J. L. Savage, referred to by Mr. Abrams. On page 154, in discussing the concrete temperature of Boulder Dam, it is stated:

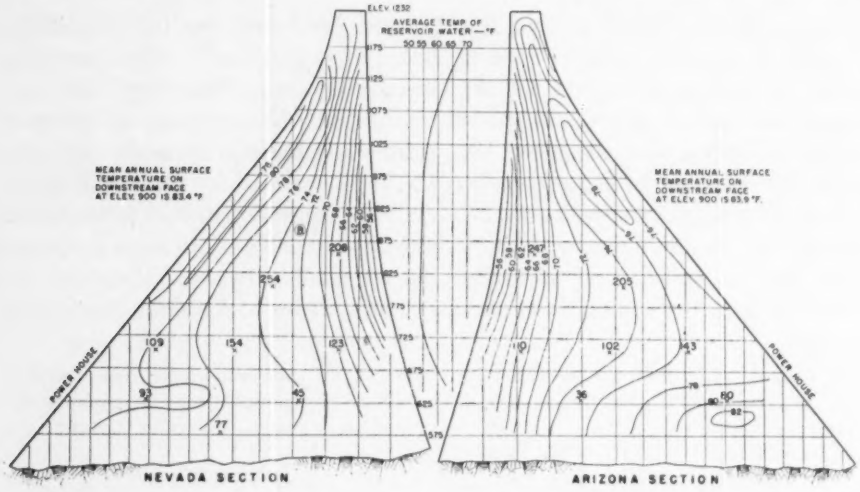
In the latter case (with the use of low-heat cement) the maximum temperature occurred at the time cooling was started, which was within one month after the concrete was placed, and the temperature was thus prevented from rising to its otherwise ultimate value, estimated to be about 9 degrees higher.

Laboratory data obtained since the early estimates were made indicate that the temperature rise that might be expected after 28 days is about 15 degrees. The data from Boulder Dam prove that the original estimate of the amount of heat that would be developed after cooling was stopped was too low, the actual average rise being about 20 degrees. These same data also prove that Mr. Abrams' estimate of 64 degrees rise is much too high.

The temperatures of the concrete of Boulder Dam as of March 1945 as determined by 396 resistance thermometers are as shown on the isothermal charts of Fig. C. The majority of resistance thermometers embedded in Boulder Dam are located in two vertical sections roughly 100 feet on either side of the central plane. The sections are shown as viewed from the central plane. In addition more detailed data are included in Table A from representative thermometers whose locations are spotted on the sections of Fig. C. The thermometers selected include No. 77 referred to in Mr. Abrams' discussion.

Fig. D shows the temperature rise since age 6 months of representative thermometers in the interior of Boulder Dam. Six months age was used as the reference point for all thermometers because in all instances cooling had been completed by that time and the concrete temperature then was at or very close to the minimum recorded. Curve A shows the average temperature rise indicated by the first 14 selected thermometers listed in Table A.

The leads from all thermometers embedded in the concrete extend to galleries within the dam. Nearly all thermometers were therefore located from 40 to 75 ft. from some gallery so that the leads might not be excessively long. Some instruments located in the region marked B on the Nevada section of Fig. C are the only thermometers in the dam which


ELEV 1232
AVERAGE TEMP OF RESERVOIR WATER — °F
50 55 60 65 70
MEAN ANNUAL SURFACE TEMPERATURE ON DOWNSTREAM FACE AT ELEV 900 IS 63.4 °F.
MEAN ANNUAL SURFACE TEMPERATURE ON DOWNSTREAM FACE AT ELEV 900 IS 63.9 °F.
POWER HOUSE
POWER HOUSE
NEVADA SECTION
ARIZONA SECTION

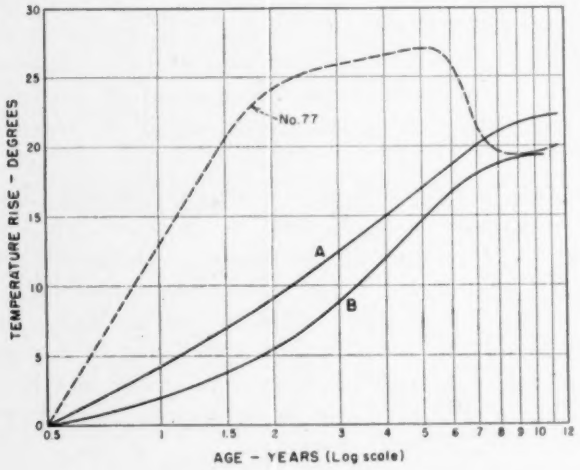


Fig. C (above)—Sections of Boulder Dam showing temperature of concrete as of March, 1945.

Fig. D (left)—Temperature rise of representative thermometers embedded in Boulder Dam.

are more than 100 ft. from a gallery or from any other exposed surface. The average temperature rise indicated by 6 of these thermometers between elevations 880 and 900 is shown by curve B of Fig. D. The temperature data of these six thermometers are also listed in Table A.

Although behavior of thermometer No. 77 cannot be considered representative of interior concrete temperatures because of its proximity to the foundation rock, it is included in the list because of the previous reference to it. This instrument is within the inner gorge of the canyon and its record indicates the influence that the rock temperatures have had on the concrete at that location. Normal portland cement was used in the construction of the lower 80 ft. of Boulder Dam which in-

TABLE A—TEMPERATURE OF CONCRETE OF BOULDER DAM

(Records from selected thermometers embedded in the concrete)
(Temperatures in degrees F.)

Therm. No.	Age—Years											
	0.5	1.5	2.5	3.5	4.5	5.5	6.5	7.5	8.5	9.5	10.5	11.5
36	48.5	54.0	59.3	63.0	65.8	68.2	70.7	72.3	73.5	74.3	75.0	75.2
80	56.8	63.8	67.7	70.4	72.5	74.3	76.2	77.5	78.5	78.8	79.0	79.1
102	49.0	56.2	59.8	64.0	66.9	69.1	71.3	72.8	73.8	73.9	74.5	
110	43.9	52.4	56.4	60.2	62.8	64.3	66.3	67.4	67.9	68.0	68.0	
143	57.8	64.7	67.3	68.8	70.2	71.6	73.0	74.0	74.9	75.4	75.7	
205	54.7	59.3	61.4	63.8	66.0	68.0	70.0	70.7	71.4	72.2	72.9	
247	44.0	51.2	57.8	61.1	63.0	64.1	64.8	65.5	65.5	65.6	65.7	
45	47.5	55.9	60.7	64.4	67.4	69.5	71.3	73.2	73.9	74.3	74.5	74.7
93	56.4	64.5	68.8	71.0	72.5	73.6	74.9	77.0	78.0	78.2	78.2	78.2
109	60.0	67.2	68.5	69.7	71.2	72.3	73.4	75.6	76.0	76.3	76.5	
123	45.3	53.2	58.5	62.1	64.8	65.8	67.0	69.5	70.1	70.0	70.0	
154	52.7	58.6	62.5	65.7	67.5	68.7	70.0	72.5	73.3	74.0	74.6	
208	44.7	52.2	58.5	61.8	63.3	64.3	65.0	66.0	66.0	65.8	65.5	
254	52.3	58.2	60.8	63.7	66.3	68.7	70.3	71.6	72.4	73.0	73.6	
204	51.5	55.6	58.9	62.5	65.3	67.9	69.4	70.8	71.2	71.7	71.8	
228	52.8	57.2	60.8	64.2	66.8	68.8	70.0	71.0	72.0	71.8	71.0	
278	52.3	55.8	59.2	62.5	65.4	67.9	70.0	71.1	72.0	72.6	72.6	
289	50.5	55.5	59.4	63.0	65.5	68.0	69.3	70.7	70.8	71.0	71.0	
327	53.7	57.0	60.2	63.6	66.4	68.4	70.0	71.0	71.2	71.5	71.5	
367	55.1	57.3	59.9	63.4	66.2	68.8	70.7	71.9	72.6	73.0	73.3	
77	55.1	75.6	80.5	81.2	81.9	82.0	79.0	75.1	75.0	74.3	74.8	75.2

cludes the location of this thermometer. However, the effect of type of cement cannot be discerned in the temperature-rise behavior of this instrument. For comparison with curves A and B, the temperature rise of No. 77 is also shown on Fig. D.

Curve A of Fig. D shows a temperature rise very close to a straight line on the semilogarithmic plot up to age 8 years, but definite deviation from the straight line after that time. Comparison with curve B indicates that some of the temperature rise in the first year after cooling may have been due to heat gained by exposure from the galleries. For the first year or two curve B is considered to be more representative of the temperature rise of interior concrete uninfluenced by surface exposure conditions. Both curves A and B show that close-to-equilibrium temperatures are established at the present time in Boulder Dam and that the temperature rise is no longer in accordance with any "logarithmic law".

Underestimates were made of the temperature of the water in the reservoir and of the average exposure temperature at the downstream face of Boulder Dam by about the same amount that the temperature rise of the concrete following cooling was underestimated. The mean

annual temperature of the reservoir water at various levels is shown in Fig. C. Direct sunlight on the downstream face has the effect of maintaining the surface at a mean annual temperature of about 84 F. instead of at the mean annual air temperature which is 72.5 F. Some of the temperature rise since cooling was stopped must therefore have been due to heat gained from exposed surfaces and cannot all have been due to continued hydration of the cement.

Boulder Dam was the first dam in which artificial cooling was used in other than an experimental fashion. The primary purpose was to cool the concrete during the construction period to what was estimated to be the final stable temperature of the dam. At the end of the cooling period the joints were expected to be opened to their widest extent. The contraction joints were then grouted and it was anticipated that the temperature changes subsequent to grouting would be small and would take place slowly over a long period. The temperature rise from the time the joints were grouted to March 1945 averages about 20 degrees. It was desired to grout the joints with the concrete subcooled and it is considered that the amount of subcooling that was actually obtained is not excessive. Since then artificial cooling has been used to some extent in the construction of at least 16 other large dams. In later instances, notably in the construction of Friant Dam by the Bureau of Reclamation and of Fontana Dam by the Tennessee Valley Authority, measures were taken to control by artificial cooling the placing temperature and the temperature rise of the concrete following placement so that fewer contraction joints would be required with consequent saving in construction costs.

In Mr. Abrams' discussion of "Heat Lost in Construction", he misinterprets the statement included in B. W. Steele's article on "Cooling Boulder Dam Concrete" that the heat lost in the interval between the placing of 5-ft. lifts, which averaged 4.7 days, was roughly one-third of the total heat generated. Mr. Abrams considers that Mr. Steele obviously referred to the amount of heat generated in 4.7 days. That is not the case. The meaning was, as stated, that during the time the top of the lift was exposed, an amount of heat would be lost from the concrete that roughly approximates one-third of the total heat that would ultimately be generated. Engineers of the Bureau of Reclamation consider that the ultimate heat development is a finite amount and estimate of that amount is based upon laboratory determination and an exponential equation of the form of the writer's equation (12). This type of equation is quite different from the logarithmic equation used by Mr. Abrams. It is understandable that Mr. Abrams does not think in terms of total amount of heat to be generated since the "logarithmic law" which he uses implies that there is no limit to the total amount of heat.

Exception must be taken to one other positive statement made by Mr. Abrams. In discussing the heat absorbed from the foundation rock he states:

The foundation rock is permanently at about 100 F. This is about mean *sun* temperature. It is obvious that concrete cannot remain at a temperature of 40 to 60 F. when it is in contact with an unlimited volume of rock at 100 F.

Only in a relatively small region of the foundation contact, where hot springs were encountered at the base of the dam, were temperatures as high as 100F recorded. The hottest water encountered during the construction period had a temperature of 114F. This was from a hole drilled from one of the diversion tunnels. The hottest water that flowed from any of the foundation drains extending to the drainage galleries of the dam was 103F. In holes drilled just outside the excavation limits of the dam at various levels for the purpose of measuring rock temperatures, average temperatures of about 80F were recorded. These holes varied in depth from 5 to 75 ft. It is not clear what Mr. Abrams means by "mean sun temperature". If he means the mean temperature of the rock surface that is exposed to direct sunlight, which might be considered the effective mean exposure temperature, then all available evidence indicates that 100F is too high a figure. Average surface temperatures measured by recording thermometers at two points on the downstream face of Boulder Dam over an 8-year period are 83.4 and 83.9F. A large amount of direct solar radiation was received at these locations. The average of the mean annual temperatures observed in three holes drilled in the rock to 5 ft. depths for the purpose of measuring rock temperatures was 76.3F.

It is reasonable to believe that rock temperatures beneath the dam will eventually be lowered due to contact of the cold reservoir water on the upstream face of the dam and on the rock surfaces upstream from the dam rather than that the concrete temperatures will eventually be raised by contact with the rock to the value of the rock temperatures before the reservoir existed. Warm water flowing from foundation drains extending to galleries within the dam has decreased in temperature since the reservoir was filled. The maximum temperature of water flowing from foundation drains at present is 80F. It is therefore expected that the concrete temperatures at the base of the dam near the rock contact will gradually be lowered rather than that they will continue to rise.

The reduction in temperature of thermometer No. 77 by about seven degrees between the fifth and the eighth year, as indicated in Fig. D, is unexplained, though a similar reduction in temperature was recorded by several other thermometers in this region. A possible explanation is that supplemental foundation grouting and drilling of additional drain holes may also have altered the temperature condition of this hot-spring area.

In conclusion, the temperature conditions that exist at Boulder Dam are sufficiently close to the estimates made by the designers to justify confidence in the methods employed to control the subsequent volume change of the concrete by controlling the temperature, removing the excess heat, and grouting the contraction joints during the construction period.

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Concrete Curing Methods*

ASTM STANDARDS

FOREWORD

Through the courtesy of the American Society for Testing Materials, which, in deference to ACI activity in the realm of field practice, withdrew its standards for curing portland cement concrete, (under the original jurisdiction of the ASTM Committee C-9, Concrete and Concrete Aggregates) ACI here republishes the latest ASTM standards on this subject, as interim information pending the completion of the work of ACI Committee 612, Recommended Practice for Curing Concrete.

STANDARD METHOD OF CURING PORTLAND-CEMENT CONCRETE SLABS WITH WET COVERINGS†

Scope

1. This method covers the procedure for curing exposed portland-cement concrete slabs by maintaining the surface moisture by means of wet coverings.

Burlap coverings

2. After the final finishing operation the concrete shall be covered for not less than 18 hr. by a double thickness of burlap kept saturated with water. The burlap shall weigh when dry at least 7 oz. per sq. yd. It shall be handled in such a manner that contact with earth or other deleterious substances will be prevented. Burlap which has become contaminated by earth or other deleterious substances shall be washed clean before use.

Protection of surface

3. Immediately after the removal of the burlap the surface of the slab shall be protected by one of the following methods:

(*a*) *Ponding*—The surface of the slab shall be kept covered with water for 7 days after the placing of the concrete.

(*b*) *Wet earth, sand, or sawdust*—A 1-in. layer of thoroughly wet earth, sand, or sawdust shall be maintained on the surface of the concrete for 7 days after placing of the concrete. The earth shall be free from large lumps.

*ASTM Designations C84-36; C83-38; C82-38; C81-36; C80-34.
†ASTM Designation: C84-36, adopted 1934; revised, 1936.

(*c*) *Wet straw or hay*—A 6-in. layer of thoroughly wet straw, hay, or similar material shall be maintained on the surface of the concrete for 7 days after placing of the concrete.

Protection of sides

4. The sides of a slab exposed by the removal of the forms shall be protected immediately after the form removal in such a manner as to give these surfaces curing treatment equivalent to that provided by one of the methods prescribed for the surface.

Removal of curing material

5. At the end of the curing period the covering material shall be removed and disposed of as provided in the contract. Straw or other material shall not be burned upon the surface of the concrete.

STANDARD SPECIFICATIONS FOR CURING PORTLAND-CEMENT CONCRETE SLABS BY SURFACE APPLICATION OF CALCIUM CHLORIDE*

Scope

1. These specifications cover the method of curing exposed portland-cement concrete slabs by maintaining the surface moisture of the concrete by means of surface application of calcium chloride.

Calcium chloride

2. The calcium chloride shall conform to the Standard Specifications for Calcium Chloride (ASTM Designation: D 98) of the American Society for Testing Materials.

Burlap covering

3. After the final finishing operation the concrete shall be covered by a double thickness of burlap, kept saturated with water. The burlap shall remain until the concrete will bear the weight of the workman without damage but in no case less than 12 hr. The burlap shall weigh when dry at least 7 oz. per sq. yd. It shall be handled in such a manner that contact with earth or other deleterious substances will be prevented. Burlap which becomes contaminated with earth or other deleterious substances shall be washed clean before use.

Application of calcium chloride

4. (*a*) After removal of the burlap, the pavement shall be sprinkled with water and 1.5 lb. of calcium chloride shall be immediately applied to each square yard of the pavement surface. The material shall be spread by means of a squeegee or suitable mechanical device, and shall

*ASTM Designation: C83-38, issued as tentative, 1931; adopted, 1934; revised, 1936, 1938.

be screeded or belted upon dissolution to insure uniform coverage of the concrete slab.

(*b*) All lumps of calcium chloride shall be broken up and distributed uniformly or removed. Calcium chloride shall not be applied during a rain, and if a rain occurs within 2 hr. after placing, the calcium chloride shall be replaced.

Protection of slab edge

5. The sides of a slab exposed by the removal of the forms shall be protected immediately after the form removal in such a manner as to give these surfaces curing treatment equivalent to that provided by the methods prescribed for the surface.

STANDARD SPECIFICATIONS FOR CURING PORTLAND-CEMENT CONCRETE WITH CALCIUM CHLORIDE ADMIXTURE*

Scope

1. These specifications cover the method of curing portland-cement concrete by accelerating the setting and hardening of the concrete by means of calcium chloride incorporated with the mixture.

Calcium chloride

2. The calcium chloride shall conform to the Standard Specifications for Calcium Chloride (ASTM Designation: D 98) of the American Society for Testing Materials.†

Use of calcium chloride

3. Calcium chloride may be added to the mix in either the dry or solution form, as follows:

(*a*) When used in the dry form from 1 to 2 lb. of calcium chloride per bag of cement, according to the temperature‡ prevailing at the time of concreting, shall be placed in the skip with the aggregates but not in contact with the cement, just prior to discharging the contents of the skip into the mixer drum.

(*b*) When used in the solution form the following procedure shall be followed in the preparation and addition of the solution to the concrete mix: Dissolve one bag of calcium chloride (100 lb.) in a quantity of water sufficient to make 25 gal. of solution (100 qt.). The solution shall be thoroughly stirred until it is of uniform concentration. From 1 to 2 qt. of this solution per bag of cement, according to the temperature‡ prevailing at the time of concreting, shall be introduced into the drum

*ASTM Designation: C82-38, issued as tentative, 1931; adopted, 1934; revised, 1938.
†1936 Book of ASTM Standards, Part II, p. 1011.
‡The following amounts are recommended: At temperatures below 80 F. (27 C.), 2 lb.; 80 to 90 F. (27 to 32 C.), 1.5 lb.; above 90 F. (32 C.), 1 lb. (1 qt. of solution contains 1 lb. of calcium chloride).

of the mixer with the mixing water. The quantity of mixing water shall be reduced by the amount of calcium chloride solution used. A positive method shall be used for introducing the calcium chloride solution.

Finishing concrete

4. The concrete shall be finished and edged promptly after placing, as concrete containing calcium chloride hardens more rapidly than plain concrete.

Burlap covering

5. After the final finishing operation the concrete shall be covered for not less than 24 hr. by a double thickness of burlap, kept saturated with water. The burlap shall weigh when dry at least 7 oz. per sq. yd. It shall be handled in such a manner that contact with earth or other deleterious substances will be prevented. Burlap which becomes contaminated with earth or other deleterious substances shall be washed clean before use.

STANDARD METHOD OF CURING PORTLAND-CEMENT CONCRETE SLABS WITH BITUMINOUS COVERINGS*

Scope

1. This method covers the procedures for curing exposed portland-cement concrete slabs, by maintaining the surface moisture of the concrete by means of a bituminous sealing coat.

Material

2. The bituminous curing material shall be of such consistency at normal temperature that it can be forced through an atomizing nozzle and applied to the concrete as a fine mist. Within 2 hr. after application it shall form a water-impermeable film and shall adhere strongly to the concrete.

Procedure—alternate method A†

3. (*a*) *Wet burlap covering*—After the final finishing operation the concrete shall be covered for not less than 24 hr. by a double thickness of burlap, kept saturated with water. The burlap shall weigh when dry at least 7 oz. per sq. yd. It shall be handled in such a manner that contact with earth or other deleterious substances will be prevented. Burlap which becomes contaminated with earth or other deleterious substances shall be washed clean before use.

(*b*) *Bituminous covering*—After the removal of the burlap and as soon as the free surface water has disappeared, and before the concrete shows

*ASTM Designation: C81-36, adopted, 1934; revised, 1936.
†The engineer shall specify the method that is to be used.

any signs of excessive drying as evidenced by lighter surface color or by the appearance of surface cracks or checks, the curing material shall be applied as a fine mist or spray by means of an approved pressure sprayer, in such a manner as to provide a continuous, uniform, water-impermeable film without marring the surface. The rate of application shall be not less than . . . . . nor more than . . . . . gal. per sq. yd.*

Procedure—alternate method B†

4. (*a*) *Bituminous covering*—As soon as possible after the free surface water has disappeared, and before the concrete shows any signs of excessive drying as evidenced by lighter surface color or by the appearance of surface cracks or checks, the curing material shall be applied as a fine mist or spray by means of an approved pressure sprayer in such a manner as to provide a continuous, uniform, water-impermeable film without marring the surface. The rate of application shall be not less than . . . . . nor more than . . . . . gal. per sq. yd.*

(*b*) *Wet burlap covering*—If for any reason the application of the bituminous curing material to any portion of the concrete surface cannot proceed at the proper time, that portion shall be covered immediately with a double thickness of wet burlap which shall be kept wet until the concrete has set.

STANDARD SPECIFICATIONS FOR CURING PORTLAND-CEMENT CONCRETE‡

Scope

1. These specifications cover the curing of concrete for all purposes including necessary precautions during the curing period in cold weather.

Strength

2. The concrete shall be so cured that the compressive([1]) or flexural([1]) strengths of specimens of the concrete 28 days old, are not less than 90 per cent of the strengths of 28-day-old specimens of the same concrete cured in moist air at a constant temperature of 70 F.

Curing methods

3. The concrete shall be cured[(2)] by any one of the following methods of the American Society for Testing Materials as specified by the engineer as suitable for the conditions of the work:

*The engineer shall insert the limiting values applying on specific contracts, depending upon the composition of the particular curing agent or agents to be used.

†The engineer shall specify the method that is to be used.

‡ASTM Designation: C80-34, adopted 1934

[1] Either compression or flexure should be selected according to whether the concrete is designed for compression or bending stresses.

[2] For a resumé of the current knowledge concerning the applicability and limitations of the various curing methods, see the report of Subcommittee XIII on Curing of Concrete of Committee C-9 on Concrete and Concrete Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 31, Part I, p. 331 (1931).

(*a*) *Bituminous coverings*—Standard Method of Curing Portland-Cement Concrete Slabs with Bituminous Coverings (ASTM Designation: C81).[3]

(*b*) *Calcium chloride admixture*—Standard Method of Curing Portland-Cement Concrete with Calcium Chloride Admixture (ASTM Designation: C82).[4]

(*c*) *Calcium chloride application*—Standard Method of Curing Portland-Cement Concrete Slabs by Surface Application of Calcium Chloride (ASTM Designation: C83).[5] and,

(*d*) *Wet coverings*—Standard Method of Curing Portland-Cement Concrete Slabs with Wet Coverings (ASTM Designation: C84).[6]

Curing during cold weather

4. (*a*) *Protected concrete*—Protected concrete when deposited shall have a temperature of not less than 50 F. nor more than 100 F. The freshly-deposited concrete and the surrounding air shall be maintained at a temperature of not less than 50 F. nor more than 100 F., until the concrete has attained 80 percent of the strength for which the concrete was designed. The methods of protection and heating shall be such as to prevent evaporation of moisture from the concrete and injury to the surface during the period of protection. The temperature of the concrete shall be observed and the concrete examined for the presence of sufficient moisture at least twice each day.

(*b*) *Exposed concrete*—Exposed concrete for which the surrounding air cannot be artificially heated shall not be placed when the air temperature is lower than 40 F. The concrete when deposited shall have a temperature of not less than 50 F. nor more than 100 F. When there is a likelihood of freezing temperature prevailing during the curing period the concrete shall be protected by means of an insulating covering of sufficient thickness to prevent freezing until the concrete has attained 80 percent of the strength for which the concrete is designed.

Preparation of specimens

5. All specimens shall be submerged in water for 24 hr. just prior to being tested.

Method of testing

6. Test specimens, in so far as applicable, shall be made and tested in accordance with the following methods of the American Society for Testing Materials:

[3] 1939 Book of ASTM Standards, Part II, p. 350.
[4] *Ibid.*, p. 352.
[5] *Ibid.*, p. 354.
[6] *Ibid.*, p. 355.

(*a*) *Compression specimens*—Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field (ASTM Designation: C31).[7]

(*b*) *Compression test*—Standard Method of Test for Compressive Strength of Concrete (ASTM Designation: C39).[8]

(*c*) *Flexure test*—Standard Method of Test for Flexural Strength of Concrete (Laboratory Method Using Simple Beam with Third Point Loading) (ASTM Designation: C78).[9]

[7] 1939 Book of ASTM Standards, Part II, p. 324.
[8] *Ibid.*, p. 328.
[9] *Ibid.*, p. 339.

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Vol. 16 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN February 1945

Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1944 to June 1945.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Flexible Joints for Concrete Pipes and Troughs (40-149)*

By F. L. FITZPATRICK

In my notes on this subject, upon which Mr. Loving comments, I did not say (or mean to imply) that rigid cement mortar joints were unsatisfactory in all cases in concrete pipe sewers and drains.

They are in many cases quite satisfactory, but as every sewerage engineer knows, they do not entirely prevent infiltration in long mains in wet ground. This applies not only to concrete pipes but to other pipes such for example as clay pipes.

Mr Loving suggests that there is a substantially different seasonal temperature variable in the cases of liquids contained in sewers as compared with water mains.

This I think is not so. The sewerage flowing in the sewer mains is almost entirely composed of the water from the water mains and its temperatures must follow closely any seasonal variation in the water temperature.

However the temperature variations are not the only factor. Sewers are often constructed through ground that varies in nature at points

†F. L. Fitzpatrick, Melbourne, Australia, ACI JOURNAL, Feb. 1944, p. 321; and M. W. Loving Secretary, American Concrete Pipe Association, Chicago, ACI JOURNAL, Apr. 1944, p. 473.

along the same pipe line. A sewer may pass through clay, then through water-bearing sand and so on. The depth too varies and hence the weight of filling above. At intervals heavy structures such as manholes are rigidly connected to the pipeline. The foundation is not varied accordingly.

Hence, though pipes are, as Mr. Loving says bedded carefully, it is certain that there is some tendency for slight differential settlement along the length.

I advocate flexible joints at or near manholes, in very wet ground, or wherever severe infiltration is feared. In other places rigid joints are satisfactory for sewers. For water mains it is our view here, after a considerable experience with many types of joints on concrete pressure pipes, that a flexible joint between pipe lengths is the most satisfactory, for a concrete pipeline.

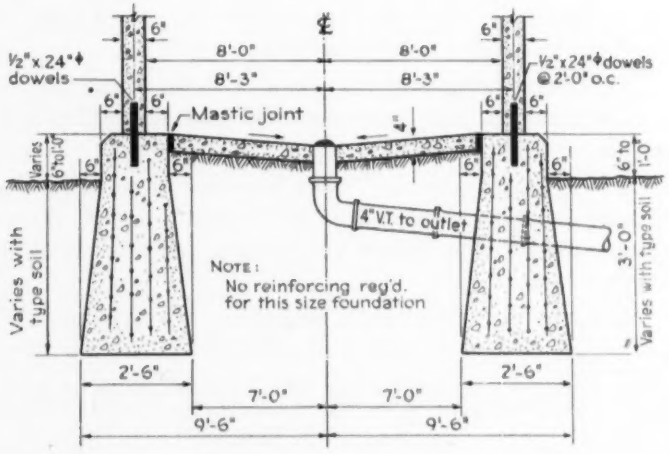
Keying or Doweling Concrete Silo Walls to Foundations—Other Silo Details (41-163)

Q—I am doing some work in monolithic concrete silo design and construction. I have read the report of your Committee 714 "Proposed Recommended Practice for the Construction of Concrete Farm Silos" with a great deal of interest.

But there is no mention made of keyways, dowels or any method of acquiring bond between foundation and the circular wall.

In building construction we usually put in a 2 x 4 in. wedge shaped keyway, but I believe on a 6-in. thickness this is not so good; dowels spaced about 18 to 24 in. apart would be more suitable. The sketch shows the major details including a provision for drainage (Fig. 1). I'd be glad to have your suggestions.

Fig. 1



By K. W. PAXTON*

A—Strange as it may seem, no anchorages or keyways are used to bind the silo proper to the foundation. There are three principal types of masonry silos used – concrete stave, tile or block and the monolithic. On the first two types it is impractical to anchor the silos and it has proven unnecessary to do so. The weight of the structure itself is sufficient. On the monolithic, while it is practical to insert dowels, they are not necessary and never used. The inside surfaces of the concrete stave and the monolithic silos are usually treated with some acidproofing anyway and this would normally seal any construction joints.

As a suggestion, I would advise your correspondent that he might carry his circular wall down below the grade to a buttment, which would eliminate the necessity of a heavy concrete foundation and make his principle construction joint below grade. This would save expensive form work inasmuch as he already has the forms to build the silo. I might also mention that concrete floors in silos are seldom used any more inasmuch as they are not too satisfactory from the drainage point of view. The drain he has shown is likely to plug up quickly and not be very effective. If the floor is smoothed off with a layer of gravel, some of the excess moisture would drain right into the ground. A drain could be provided from this gravel course out to some low point.

By D. G. MILLER†

This matter was discussed in one of the committee meetings and the opinions expressed generally were to the effect that it was not necessary to tie a concrete silo onto the base although there certainly are no objections to doing so. So far as I can recall, none of the concrete stave silos of Minnesota is keyed in any way to the foundation. I never have seen a concrete silo that had moved on its base although I have been told that such a thing actually did happen in one case during an extreme windstorm.

I have been interested in your correspondent's detail showing the foundation for a 16x50-foot monolithic silo. This plan looks fine except for the one detail of the floor drain. Rather than slope the floor in all directions toward an outlet at the center of the floor we believe there should be a loosely covered trough cast entirely across one diameter of the floor. The accompanying detail (Fig. 2, next page) shows the floor plan of an experimental silo constructed under the direction of the Division of Agricultural Engineering of the University of Minnesota. This silo was filled with grass the past three seasons and this year is being

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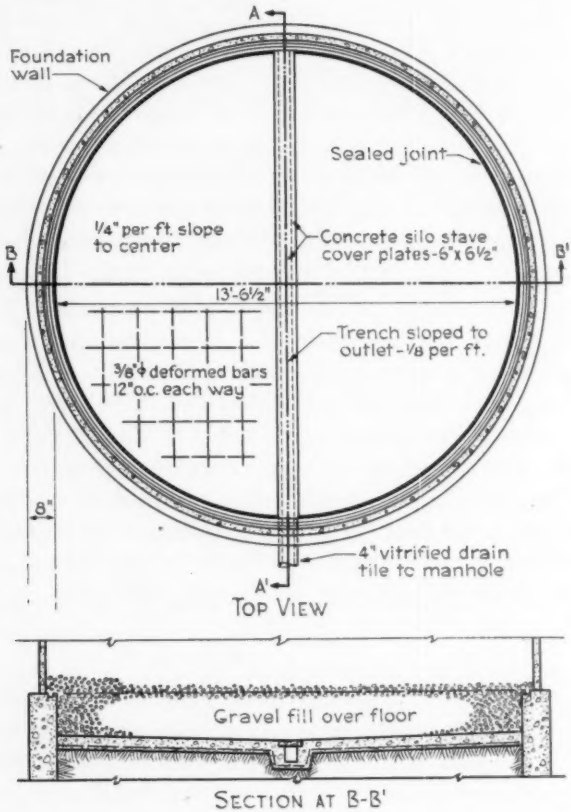
filled with green corn. The floor drain has functioned perfectly and one year carried around 5000 gallons of silage liquids.

Fig. 2



Setting Heavy Machinery on Concrete Bases (41-164)

Q—The literature of setting heavy machinery on concrete bases is scanty—what information are you able to supply? It is a ticklish problem. What about the protection of the metal in contact with the concrete?

By I. L. TYLER*

A—It is hardly to be wondered that manufacturers of heavy equipment are finally becoming concerned about material and procedures for grouting machinery bases. The question (except perhaps on protection of metal) has come up on every construction project with which I have been associated. Some of them have been at least partially answered.

*Portland Cement Assn., Chicago

Heavy machines (turbines, generators, motors, compressors and the like) resting on concrete bases are usually placed accurately in position on shims and wedges or sometimes jacks which remain in place as concrete or mortar is placed to fill the gap between the original concrete and the machine base. It is important that the concrete or mortar be of adequate strength and that it fills the space completely.

Since the weight of the machine is supported temporarily on a comparatively small area of metal, the amount of movement necessary to permit full bearing of baseplate against supporting mortar must be very small if original alignment of equipment is to be maintained and disrupting effects of vibration avoided. While it is seldom, if ever, possible to obtain perfect contact between mortar and the machine base, it is possible to approach such a condition by use of proper materials and procedure.

One very satisfactory method is that of "dry packing" portland cement mortar into the space between concrete and the machine base. It is applicable only under conditions of good accessibility. By this method a high quality mortar which will shrink very little on drying may be used.

For inaccessible locations a mortar which will flow into place is required, though in every case the very least possible amount of mixing water should be used. With such mortar, settlement (bleeding) may be very appreciable before hardening takes place, leaving a space between the machine base and the mortar unless remedial measures are used. To prevent this settlement a small amount of aluminum powder (of the order of one to two grams per bag of cement) may be mixed with the mortar. Simple preliminary field tests should be made using aluminum powder in mortars containing the job cement to determine the amount of aluminum powder needed. A quantity just sufficient to produce a slight expansion should be used.

Special mixtures of cement, iron, and other ingredients have been used with varying success, but my preference is for a good quality cement mortar with the correct amount of aluminum powder added.

Shrinkage during drying may be of some importance where thickness of the mortar is considerable. Since shrinkage is directly affected by the quantity of water in the mix, the smallest amount of water consistent with placeability is desirable from this as well as strength and other standpoints.

Cement-to-sand proportions between 1:1 and 1:3 by weight have been used for mortar under machine foundations with 1:2 or 1:2½ predominating.

It is often advantageous to introduce fluid mortar through a pipe using several feet of hydraulic head to force the mortar into position.

Rodding the pipe filled with mortar will add to the effective hydraulic head and permit use of a reasonably stiff mix. Vibration may be of advantage also, but machinery erectors object to its use because of the danger of shifting accurately aligned equipment.

The old concrete should be carefully cleaned before any of the heavy equipment is put in place. Usual practice is to chip the surface with an air hammer. Then it must be kept clean until the machinery is assembled and that is usually difficult to do because oil and grease are always present during erection. Care in this respect is necessary because the surface is inaccessible for thorough cleaning after assembly has started. The old concrete should be damp but with no water standing on its surface when the mortar is placed. Exposed mortar should be moist-cured for at least 3 days.

In preparing erection plans consideration should be given to accessibility if satisfactory grouting is to be expected.

The question of satisfactory surface coatings for protecting metal embedded in mortar is one that I have not encountered. It would be my impression that a good grade of mortar would give about as satisfactory protection as most other materials.

[The letter from which the following is an excerpt combines question and discussion of that question by Ralph Lewis, Power Engineer in charge of the Broken Hill Associated Smelters, Port Pirie, South Australia sent to ACI by T. J. Cavanagh of the Australian Cement Mfrs. Assn. —EDITOR].

By RALPH LEWIS

What constitutes the better practice when erecting heavy oil engines, or any other machinery for that matter: leaving the wedges (or steel strips) in after grouting, or withdrawing them and allowing the engine base to rest on the grout, assuming that the engine is erected on concrete foundations and not on steel structures.

Years ago I installed a considerable number of heavy gas and oil engines. The bases were usually lined up on steel wedges, but in some cases on parallel strips and shims, leaving about ¾-in. space for grouting. A great deal of care was taken with the grouting, but when it was completed and well dried out the wedges or strips were withdrawn, leaving the engine base supported by the grout only. No trouble was ever experienced with these jobs. I have followed this method with all of our equipment in the power plant here at Port Pirie—turbines, air compressors etc. We have three Belliss Morcom air compressors which have been running for the greater part of 24 hours per day and seven days per week for more than twenty years. We have never had a warm bearing and the shafts have never been lifted.

There is a considerable trend of opinion now in favor of leaving the steel strips under the engine after grouting in. Why?

The claim is that cement grouting will not stand the racket, and that the weight of the engine base must be taken by steel strips.

I have had opportunities to examine foundations after old engines had been dismantled. In cases where wedges had been left in, the edge of the cast iron base had worn into the wedge until the grout had taken the weight of the base. When parallel steel strips had been used the indications also were that the engine base had been down to the grout. Accepting the fact that there is some shrinkage of grout when drying, one can only assume that the parallel steel strips, owing to the weight being localized on them, had been forced into the concrete until the grout had taken the weight of the engine bed.

Here is an actual case. An oil engine of 600-700 b.h.p. and the machined portion on the underside of the base which, if no wedges or strips were used and left in, would rest on the grouting, had a total area of thirty square feet. As the completed job weighed around thirty six tons, the pressure on the grouting would have been 2688 lb. per square foot of bearing surface.

It was, however, erected on 32 steel strips or packing pieces, each strip having an area 9 by 3 in. The total area of all of these strips was therefore 6 square feet. When this job was grouted in (and again accepting the fact that the grout would shrink slightly and leave the weight on the strips), the pressure per square foot of strip area would be 13440 lb. or 6 tons.

Is it reasonable to assume that the strips would continue to carry this weight without being gradually forced down into the concrete bed until the grouting took the weight, and if the grout is eventually to take the weight, well, why leave the strips in at all?

I have had bearing trouble with engines where the strips had been left in, and I consider that it was owing to some strips being forced into the concrete more rapidly than others, thus allowing the base to be sprung out of alignment. The fact remains that I cured the trouble by withdrawing the strips, putting the base hard down on the grout, and re-bedding the shaft.

When erecting engines and heavy machinery on concrete foundations my practice has always been to leave the top of the foundations fairly rough, then locate the bed or base plate of the machine or engine, line it up and level it by means of steel wedges, leaving approximately ¾ in. between the concrete and the iron bedplate.

This space is then well grouted up with a mixture of 1:1, sometimes 1:2, of cement and clean sharp sand. After the grouting has hardened,

the steel wedges are withdrawn, so that the base or bed rests on the cement grouting, and is held there by the holding-down bolts.

I have never experienced any trouble with jobs done along these lines, but I *have* had trouble with jobs where the steel wedges have been left in.

By H. L. FLODIN*

The subject Mr. Lewis brings up, that is whether or not the wedges under grouted machine bases should be removed is one on which opinion seems to be about equally divided. There is very little published material on the subject of grouting and anchoring machine bases. A very good book was published back in 1923 by McGraw-Hill entitled "Machinery Foundations and Erection" by Terrell Croft. The book has been out of print for a good many years and it never went into a second edition. Mr. Croft had the following to say about removing wedges.

> The removal of the leveling wedges is a subject about which there has been considerable discussion. It appears that certain erecting men for some reason or other allow these wedges to remain after the bed plate has been grouted. The most prevalent opinion, however, appears to be that for most conditions the wedges should be removed. If the wedges are left in practically all of the weight of the machine and bed plate may ultimately rest on them. The grout may not be in sufficiently intimate contact with the bottom of the bed plate to assume any great portion of the vertical load. Where the wedges are left under the bed plate there is a possibility of a space of something like 1/1000 of an inch between the grout and the bed plate. Where the wedges are removed the bed plate must, of course, bear down into intimate contact with the grout. The entire weight is thereby uniformly distributed over the whole foundation top. Obviously, the grout, if it extends up within the bed plate, should prevent lateral movement whether the wedges are removed or not. For steam-turbine bed plates the general practice seems to be to permit the wedges to remain.

In another paragraph describing grouting under steam-turbine bed plates Mr. Croft states:

> The wedges used under the bed plate in aligning the turbine are not ordinarily removed but are left embedded in the grout.

We recently had occasion to interview a number of men familiar with machinery erection and their opinions on the subject were divided. In general, I should think the safest practice would be to remove the wedges and Mr. Lewis' experience would certainly indicate that removal of all wedges is the safest practice.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Control of quality of ready mixed concrete

National Ready Mixed Concrete Association, 1944. Reviewed by STANTON WALKER

A 71-page booklet, compiled by a committee of practical operating men, as a guide for the ready-mixed concrete manufacturer. Principal emphasis is placed on those factors affecting the control of uniformity of ready-mixed concrete. Comprises twelve chapters and four appendices. Among the subjects covered are control of concrete at source, batching plants and batching control, mixing and discharge control, inspection and maintenance of equipment, cold weather concrete, and methods for designing proportions. One chapter discusses strength design in relation to the theory of probability; another chapter presents a new application of the fineness modulus method of design which takes into account cement in calculating the fineness modulus. A carefully selected list of references is included in the booklet.

South American building is challenging

ARTHUR J. BOASE, *Engineering News-Record*, V. 133, No. 16 (Oct. 19, 1944), pp. 121-128
Reviewed by S. J. CHAMBERLIN

This is the first of a series of articles planned to make available the detailed methods of design and the code provisions that permit bold use of concrete. Building activity is great in Brazil, Argentina and Uruguay. The general type of construction consists of reinforced concrete frame and floors with masonry walls. Structural steel is scarce and high in price, but the "concrete consciousness" probably springs from the European background of the early designers and such men as Baumgart. The engineers are constant students and indeterminate design methods are universally used. Building codes are favorable to concrete. Material costs are high as indicated in a table. Construction labor costs are also listed. The cost of concrete in Brazil in a building frame is about $40.00 per cubic yard with about 20 percent of this amount going for labor.

Resurfacing concrete slabs

Report of the New Jersey State Highway Department (1943) HIGHWAY RESEARCH ABSTRACTS

An experiment was conducted in one county in connection with the disintegration of concrete slabs caused by the chemical action of salt used as an abrasive in ice-control operations.

Experiments on certain slabs indicated that it was possible to renew the surface with a thickness of not more than one-half inch, by thoroughly cleaning the pavement with

compressed air, applying a grout of cement and allowing this to take its first set, and then adding a mixture of one part cement, one part sand and two parts half-inch clean stone.

It is believed that this will prevent further disintegration of concrete pavements due to the chemical action of winter abrasives. It may even provide a new service by the Department to city and local road authorities.

Circular reinforced concrete footings

ROBERT W. ABBETT, *Civil Engineering*, V. 14, No. 9, Sept., 1944, pp. 401-402.
Reviewed by J. R. SHANK

The author calls attention to the principle that bending stresses in footing slabs are distributed along radial and circumferential lines with respect to the column load. This, he says, is true for square footings as well as for round ones. He gives a formula for the bending moment, taking Poisson's ratio into account.

$$M = \frac{PR}{4}(2-3a+a_3)$$

Where P = total column load
R = radius of footing
A = the ratio, column radius over R

In the case of square columns, moments may be computed on the basis of a round column of equivalent area.

Support for a tall stack on poor soil

CLARENCE W. DUNHAM, *Engineering News-Record*, V. 133, No. 12 (Sept. 21, 1944), pp. 86-91
Reviewed by S. J. CHAMBERLIN

Extensive foundation studies were required before a satisfactory design could be made for the support of the 540-ft. reinforced concrete stack. The design combines a spread footing of 99 ft. in diameter to insure against unequal settlement due to the variable foundation material and removal of enough overburden to about equalize the load that the stack would have otherwise have placed on the deeper beds of soft clay. The support is essentially a large shallow "tank" 28-ft. deep, with a heavy base, a 12-in. circular wall and a slab covering at grade. The concrete protection below the membrane waterproofing, and the ring wall outside of the edge of the footing were placed first and made heavy enough to form a "pan" at the bottom of the excavation. With the bottom thus protected the 10-ft. thick slab and the other work could be placed. To reduce the cantilevered portion of the footing the lower part of the stack was belled out in a solid ring, the top of the belled portion being supported by a 24-in. slab to resist the inward thrusts at the top. The "tank" wall was supported laterally by a concrete deck.

Soil-cement paving costs reduced by use of clay marl

LT. COMDR. C. B. FRIDAY (CEC) USNR, *Engineering News-Record*, V. 133, No. 22 (Nov. 30, 1944), pp. 67-70
Reviewed by S. J. CHAMBERLIN

The selected clay marl borrow required slightly more than 9 percent of cement by volume compared with 16 percent that would have been required for the fine sandy loam at the site of the big airbase. Sufficient clay marl to provide a 6-in. compacted base course was trucked in and spread with graders. Cement in bags was spotted, dumped in piles and spread uniformly with spike-tooth harrows. Dry and moist mixing was accomplished with disk harrows, cultivators and rotary fillers, aided by a five-bottom gang plow cutting to a depth of 6 in. Primary compaction was with sheepsfoot rollers, followed by a drag harrow to break up the crust. Graders did the final shaping and were followed by drag brooms and smooth rollers, which in turn were followed by pneumatic

tire rollers. Construction of the runways in 30-ft. lanes without side forms was made possible by leaving a partly consolidated shoulder and then cutting back the completed section a distance of 12 to 18 in. An early application of bituminous material served both as an aid to curing and as a prime coat for the wearing course. After about two weeks the treated area was surfaced with a hot bituminous mixture about 1½ in. thick.

Seven hangars with modern equipment in rigid-frame concrete structure

HOWARD J. SNITOFF, *Engineering News-Record*, V. 133, No. 10 (Sept. 7, 1944) pp. 116-120
Reviewed by S. J. CHAMBERLIN

The continuous rigid-frame building, seven hangars wide and 200 feet deep, has main interior frames consisting of seven 130-ft. spans with typical beams 5 ft. deep by 3-ft. wide. The beams deepen from a point 37 ft. from the columns to form haunches with a total depth of 10 ft. 9 in. at the column line. The main frames spaced 25 ft. c. to c. are tied at the column lines by 10 ft. 7 in. deep by 1-ft. wide cross beams and just beyond the quarter points with 16 x 16-in. tie beams. The main beams have a construction joint just beyond the quarter-point of the 130-ft. span and the first placement was to this joint on both sides of the column with one-half the length of the transverse tie placed simultaneously. I-beams on timber towers supported the forms. Center portions were placed within 24 hours and to prevent settelment, that would put a cantilever load on the green concrete previously cast, screw jacks were set in the falsework to take up any movement. Precast roof joists support precast roof slabs. The floor is an 8-in. plain concrete slab. The huge steel-framed doors open by rising about 6 in. and then rotating on a horizontal axis, the lower half projecting outside to form a canopy over the portal.

Heavy duty runway on poor ground

RHODES E. RULE and JOHN F. L. BATE, *Engineering News-Record*, V. 133, No. 20 (Nov. 16, 1944) pp. 103-106
Reviewed by S. J. CHAMBERLIN

Dredged fill under a sub-base of sand topped by a crusher-run course of sand and gravel form the support for a 12-14-in. thick portland cement concrete runway over soft ground, designed to take the load of the largest planes. The runway was laid in 25-ft. strips by three 34E single drum pavers working together, followed by a spreader with horizontal traveling blade and a vibrating screed, a full-depth internal vibrator and a dowel setting machine. After the dummy joints are made, the surface is obtained with a vibrating finishing machine followed by a hand finish operation using wood floats and a burlap drag. The full depth vibrator machine is equipped with 8 "stingers" spaced at about 3 ft. centers except at the edge where the vibrator path is about 6 in. from the side form. After the work was started it was concluded that ample vibration was provided by the full depth vibrator so vibrators on the screeds of the spreading and finishing machine are not normally operated. The carefully controlled concrete has a water-cement ratio of 0.55 by weight, an average 28-day strength of 4,000 psi. and a modulus of rupture of 700 psi.

Almost $18 billion spent on roads and streets in decade

Highway Highlights, Sept. 20, 1944. Highway Research Abstracts

Total street and highway construction and maintenance expenditures for the 10-year period, 1933-42, amounted to $17,754,000,000, according to the United States Public Roads Administration.

Construction costs totaled $8,190,000,000, with the Federal Government supplying $2,160,000,000, the States $3,030,000,000 and counties, cities and towns, $3,000,000,000.

Maintenance costs of roads and streets for the decade amounted to $5,365,000,000 with the States spending $1,931,000,000; counties, cities and towns $3,434,000,000 and Federal Work Relief $4,199,000,000.

Expenditures by years on street and highway construction and maintenance by all agencies were:

1933—$1,110,000,000	1938—$2,177,000,000
1934—$1,554,000,000	1939—$2,173,000,000
1935—$1,290,000,000	1940—$2,110,000,000
1936—$2,022,000,000	1941—$2,026,000,000
1937—$1,824,000,000	1942—$1,468,000,000

Pressure grouting railroad roadbeds

FRANK W. CAPP, *Engineering News-Record*, V. 133, No. 8 (Aug. 24, 1944) pp. 74-79
Reviewed by S. J. CHAMBERLIN

Pressure grouting with portalnd cement is being used to stabilize 100 miles of main line track on the Santa Fe Railway system in Kansas. The idea is to stabilize the existing supporting material rather than to form a structural subballast concrete slab. Free water and liquified soil are forced out and replaced with grout which seals the passages and voids against further penetration and helps distribute loads uniformly over the roadbed. Hydraulic pressure equipment and techniques have been developed from two years experience on smaller jobs of varying conditions. Bull points are driven by air-hammers and withdrawn to provide holes for the grouting points of steel tubing. The grout discharge line consists of 100 to 300 ft. of 5-ply, 1¼-in. air hose equipped with quick-acting couplings. Injection points are spaced according to conditions, usually from 20 to 80 in. Proper depth for the tip is at or near the bottom of the ballast, with pumping continuing until the grout escapes from a nearby driven point or breaks out at some other place. After the roadbed is grouted all ballast is removed to 8 in. below bottom of ties and replaced with chat ballast. Mixtures used range from 1 part cement 6 parts of sand and 7 parts of water, to 1 part cement, 8 parts of sand and 9 parts of water by volume. Fine sand is necessary in order to pump properly and to permeate the small passages and fine material in the roadbed. Costs vary considerably but one 22-mile job averaged $0.42 per foot of track. The average rate of one unit over a 16-month period was 300 ft. per day.

Airbase requires varied paving operations

EUDELL G. WHITTEN, *Engineering News-Record*, V. 133, No. 18 (Nov. 2, 1944), pp. 70-74
Reviewed by S. J. CHAMBERLIN

Portland cement concrete for taxiways and aprons, bituminous concrete for the runways and bituminous stabilized soil for the shoulders were used in enlarging an airfield. The portland cement concrete was made with air-entraining cement, 0.025 to 0.045 percent Vinsol resin being interground with the cement. Both standard pavers and truck-mounted mixers were used. At first the materials for the truck mixers were dry batched at the central plant and hauled dry with no mixing until arrival at the job where the measured mixing water was added and the concrete mixed in the vented truck mixers for 5 minutes at a peripheral speed of 225 ft. per min. Because of difficulties in transporting the water to the site, the procedure of adding the water to the drum at the plant but not mixing until the site was reached was adopted. Using the same cement, the same water-cement ratio and similar aggregates there was no significant difference in the concretes produced by standard 27-E and 34-E dual-drum pavers mixing 1½ minutes and that produced by the truck-mixers mixing for 5 to 10 minutes. The air content for the concretes produced by both methods averaged 3 percent as determined from unit weight tests of the fresh concrete.

Anchorage for reinforcement in concrete

Technical News Bulletin, National Bureau of Standards, No. 329, Sept. 1944.
HIGHWAY RESEARCH ABSTRACTS,

In designing reinforced concrete structures, it is ordinarily assumed that the tensile and compressive deformations in the steel reinforcement are the same as in the adjacent concrete. To assure an approximation of this common action of the two materials, the design stresses in the reinforcement and the bond stresses between the concrete and the reinforcement are limited to values indicated to be safe by the results of laboratory tests and experience. Often, the necessary anchorage for the reinforcement is obtained by bending the ends of the bars to form semi-circular hooks. Current specifications limit the radii of such hooks and their lengths beyond the point of curvature, but they do not include effective requirements governing the surface texture and the nature of the deformations on deformed bars. In an investigation of the factors that influence the effectiveness of hooks as anchorages for reinforcement, it has been found that the deformations on the bars have a greater effect upon the performance of hook anchorages than any of the factors now limited in specifications. The results of the investigation provide information on which to base specifications for anchorages that require less steel and are more likely to function as intended than those now in common use.

Mixing and placing concrete at Fontana

Engineering News-Record, V. 133, No. 22 (Nov. 30, 1944), pp. 75-79 Reviewed by S. J. CHAMBERLIN

Concrete for the 470 ft. high, 2,800,000-cu. yd. dam was placed in about 20 months. Job-made aggregate was carried by more than two miles of belt conveyors to the plant. The cement was blown through 6-in. lines for a distance of 3000 ft., including a 300-ft. vertical rise, to storage silos and blown from there to the bins at the mixer plant. Nearly all of the concrete was prepared in a single mixing plant housed in an octagon-shaped building containing eight bins around its outside and two cement bins in the center. Six aggregate and sand batchers, two cement batchers and a water batcher were suspended under the bins and discharged through a turnhead into any one of five 4-cu. yd. front-charge tilting mixers. Concrete was discharged through a central cone to a 42-in. conveyor belt that fed to a series of belts to take the concrete either up or down to three construction levels. The first belt had an upward angle of 13 deg. and operated at 216 ft. per min. The 4-cu. yd., bottom-dump buckets, set on flat cars, were loaded from a transfer hopper at each level. Loaded trains moved out on trestles to the full revolving and hammerhead cranes. The hammerhead cranes had special controls for adjusting the speed of raising or lowering loads to the weight handled so that for each movement the motor was loaded to its full capacity as either a motor or a generator. Cantilever forms, without hog rod stiffeners, were used to keep the lift area free of obstructions. Operations were speeded by material boxes, that contained all of the equipment to prepare a lift and a lower compartment to hold the muck from the "cutting" operation after initial set, which were set onto a block as soon as a lift was placed.

Design facilitates rapid construction of TVA's Fontana Dam

Engineering News-Record, V. 133, No. 20 (Nov. 16, 1944), pp. 97-102 Reviewed by S. J. CHAMBERLIN

Designed with speed of constructiion as the paramount factor but with scarcity of materials and shortage of labor important considerations, the 2,800,000-cu. yd. dam is approaching completion in 20 months from the start of concrete placing. The main mass section of the gravity structure is almost entirely free from any irregularities, the diversion works and permanent spillways being through rock abutments. The powerhouse is almost entirely separate although at the down-stream toe. The mass dam was divided into 35 regular blocks each 50 ft. wide. In the river channel the blocks have a height of

more than 470 ft. and a length, parallel to the river, of nearly 400 ft. Up and down stream the blocks are divided into as many as four sections by deeply keyed vertical construction joints that are grouted, after all expected shrinkage has taken place, to make each 50-ft. wide block a monolith. Coarse aggregate in four sizes, with a maximum of 6 in., and fine aggregate is of quartzite prepared from a source near the site. Mass concrete of 0.8 bbl. of Type II equilvalent cement per cu. yd. and 1 in. slump was used at first with a water-cement ratio by weight of 0.75. To speed construction the cement content was later increased to 0.85 bbl. and the slump to 2.3 in. Strengths averaged 4,300 psi. at one year. A richer mixture of 1.25 bbl. of cement per cu. yd. was used for the 5 to 8 ft. near the outside faces of the dam. The required cooling and shrinkage before grouting to unify has been attained by circulating refrigerated water through embedded pipes placed on top of the usual 5-ft. vertical lifts. An innovation in power-house design practice in this country is welding the scroll case. The spillways utilize a simple deflector scheme to throw the water upward to dissipate the energy.

Monolithic concrete seaplane hangars

ROBERT ZABOROWSKI, *Civil Engineering*, V. 14, No. 8, Aug., 1944, pp. 355-358.
Reviewed by J. R. SHANK

Twin seaplane hangars, each covering a total floor space of 413 ft. wide by 250 ft. deep, using arches and Z-D slab construction, were constructed for the Navy. The cross-section consisted of two clear main areas under the arches and three two-story leantos. The clear door openings into the hangar areas under the arches were 160 ft. The leantos were 31 ft. wide. The curved roofs of the hangars were reinforced concrete shells nominally 3½ in. thick supported by arches 35 ft. 6 in. on centers. The center five-eighths of the arches had radii of 165 ft. and the remaining radii, excepting for short transitions of 100 ft., were 65 ft. The springing lines were 28 ft. above the floor and the high points of the ceilings, 55 ft. Additional transverse stiffness was afforded for the thin shell by intermediate stiffening ribs 10 x 20 in., midway between the arches covering all of the 165 ft. radius curvature portion and part of both 65 ft. parts. There were no expansion joints in the entire 413 ft. cross-section. In the other direction the building was divided into two 71 ft. and two 54 ft. 3 in. units. Each hangar roof section was poured independently of the adjacent section and the leantos were poured well in advance of the main hangars.

The design was highly indeterminate statically having from 15 to 24 redundants. A minimum factor of safety against buckling of 6.5 existed under full load at the crown between the arch and the stiffening rib.

The amounts of materials used were equivalent to 8.39 lb. of reinforcing steel and 15.5 in. of concrete per sq. ft. of usable floor space.

The deflection at the crown at the time of decentering was 0.985 in. as compared to a calculated value ($E = 3{,}000{,}000$ psi) of 0.628 in.

How to use air-entraining cements

H. G. FARMER and G. L. LINDSAY, *Engineering News-Record*, V. 133, No. 8 (Aug. 24, 1944), pp. 64-67
Reviewed by S. J. CHAMBERLIN

There is substantial agreement that air-entraining cements can improve the resistance of concrete to scaling, and freezing and thawing with resultant improvement in durability. For best results it is necessary to control the amount of entrained air so that it will be neither deficient nor excessive. It has been concluded that added air (in addition to the 1 to 2 percent normally present) should be about 3 percent. The volume of entrained air depends not only upon the amount of entraining agent used, but also upon the characteristics of the cement and the effect upon the concrete. Tests have been devel-

oped together with field experience to determine the amount of agent to be added at the mill. It is thought that the increase in durability is obtained by the reduction in segregation and bleeding. Alkalis in the cement are dissolved and combined with the air-entraining agent to form small amounts of soap, which entraps air in uniformly distributed, microscopic bubbles with resultant increase in plasticity and workability with less water. Damage to the concrete due to freezing is reduced, partly because of the small volume of water relative to the surface of the channels and partly because of the cushioning effect of the air in the tiny voids left by the bubbles. The loss in strength is at least partially offset by necessary mixtures changes, less sand and water being needed because of the increase in volume of the mortar. The amount of sand is generally reduced about 3 to 6 percent of the total aggregate, and the water reduced about a gallon or more per sack with lean mixtures and half a gallon or less for rich mixtures. Thus the leaner mixtures may give strengths greater than with normal cements. Rich mixtures give somewhat lower strengths. If durability governs, a loss in strength may not be significant. Any loss in strength is also partially offset by the increased uniformity of results.

Commercial Vinsol resin cements

Technical News Bulletin, National Bureau of Standards, No. 330, Oct., 1944.
HIGHWAY RESEARCH ABSTRACTS

Concrete pavements have been found more resistant to scaling, to destruction by freezing and thawing, and by action of salt solutions when a small percentage of air is incorporated in the concrete when made. The commercial production of air-entraining portland cement by intergrinding small quantities of Vinsol resin in the manufacturing process is a rather recent development. The properties of some of these Vinsol resin cements, however, were such that it was impossible to test them according to the existing specifications. New tests were required as well as the development of new testing techniques in order to evaluate adequately the properties of the new material.

The Bureau recently procured 64 commercially manufactured Ninsol resin cements in order to study the water requirements of the pastes and mortars, the quantities of air entrianed in the various pastes, mortars and in concrete, and also to make a survey of strengths of the test specimens.

The neat pastes of Vinsol resin cements were stickier than the neat pastes of the untreated cements and also had a tendency to stiffen more rapidly. The cement-sand mortars made of Vinsol resin cements and proportioned according to present Federal specifications were in most cases more plastic than the mortars of the untreated cements. Whereas the neat pastes of Vinsol resin cements required as much or more water than the neat pastes of the untreated cements, the cement-sand mortars usually required less water for the same workability.

Concretes made from the different cements entrained quantities of air ranging from 1 to 14 percent. The air contents of the pastes, the 1 to 3 mortars, and the 1 to 2.75 mortars were not indicative of the air entrained in concrete. Of the various mortars and pastes tested, the 1 to 4 mortar, which had been proposed as a performance test, had the greatest range of air contents with the different cements.

Most of the Vinsol resin cement mortars had both tensile and compressive strengths greatly in excess of specification requirements. A few of the cements, however, failed to meet the 7 and 28-day compressive-strength requirements. The cements had a wide range of strengths at 3 days; for example, tensile strengths were obtained for the different cements ranging from 195 to 465 psi. and compressive strengths ranging from 900 to 3,360 psi.

Powered concrete ships of World War II

ROBERT E. MCLAUGHLIN, *Engineering News-Record*, V. 133, No. 16 (Oct. 19, 1944) pp. 94-98
Reviewed by S. J. CHAMBERLIN

The twenty-six self-propelled concrete ships constructed by one company have an overall length of 366 ft., a molded width of 54 ft. and a depth of 36 ft. amidships. There is only a single deck over most of the vessel with only ten transverse bulkheads and no longitudinal bulkheads so that space for cargo is large and unobstructed. Rigid-frame analysis is used for design throughout the entire structure. The maximum stresses permitted are listed. The shell thickness is 6½ in. reinforced with two layers of ⅞-in. dia. plain bars and two horizontal layers of 1-in. dia. plain bars. Stringers, 6½ in. wide and 18 to 20 in. deep, spaced about 4 ft. c. to c., span frame to frame and extend the entire periphery of the shell. Transverse frames are spaced 10 ft. 8 in. c. to c., through the midship portion and at 4 ft. c. to c. at the ends. The main deck slab is 5½ in. thick. Outside face of the shell has ⅞ in. and the main deck ¾ in. of concrete over the steel. In all other positions a ½-in. minimum cover is provided.

Forms for the hull bottoms, consisting of narrow width 1-in. cypress boards, were permanently attached to the basin in which the ships were built and floated. Plywood exterior forms were supported on rigidly built movable frames that were bolted to the bottom of the basin and were made in sections just a little narrower than the spacing of the frames of the ship. Reinforcing was carefully detailed and preassembled in sections with the aid of wooden jigs. The inside forms, usually of steel, were held to the outside by special ties placed through the wales that form the side of horizintal beams and stringers of the permanent concrete. The concrete is placed in three principal castings, to the 6-ft. water-line, to just below the deck and then the deck.

The new light-weight aggregate is made by crushing, drying and grinding fullers earth into a fine powder which is fed into a rotating cylinder and sprayed with water in drops of varying size. The drops of water pick up varying amounts of dry material forming small hard spheres which are calcined into vesiculated nodules. Characteristics of the aggregate are listed. The concrete has a water-cement ratio of 0.45 to 0.50, a cement content of 9.2 bags, a slump of 4½ to 6 in., and weighs from 112 to 117 lb. per cu. ft. Prepared in truck-mounted mixers that hauled to any part of the basin, the concrete was lifted to deck level by a crane, using a 1⅓-cu. yd. bucket for delivery to master containers. From the transfer hoppers the concrete was taken in buggies along a temporary platform and dumped through elephant-trunk chutes. External vibrators were spaced about 6 ft. c. to c. each way and internal vibrators were inserted into each batch as it was deposited.

Fourth Annual

TECHNICAL PROGRESS SECTION

FOR COMMERCIAL PRODUCERS

February 1945

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FOREWORD

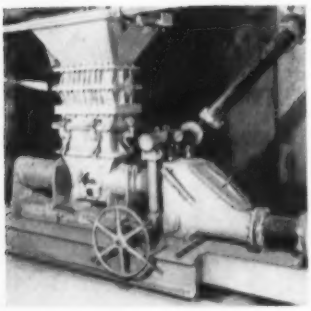
It has been gratifying to the Institute's Publications Committee to note how "producers for engineers"—producers of tools, materials and specialized services for the field of concrete—are sustaining the objects of the annual Technical Progress Section of the ACI Journal.

There is evident an increasing appreciation by our advertisers of the possibilities of advertising "copy" which is interesting, informative, constructive—advertising copy which, in avoiding over-statement, carries more conviction to minds trained to rely upon well-supported testimony. Such appreciation is not yet complete; over-zealous salesmanship persists; yet advertisers have rarely failed to co-operate in copy revisions toward more conservative and convincing claims. Rather than to exclude the somewhat over-zealous, it has been the policy of Publications Committee reviewers of advertising copy to judge its acceptability by the test: "Does it deceive?" and beyond that to depend largely upon the gradually efficacious influences of the JOURNAL readers' judgments, through comparison of the degrees of producer restraint and responsibility, believing strongly that TPS is serving in an increasingly important way in its record of important progress in concrete construction and manufacturing practice.

Institute endorsement of the pages which follow in this section is neither given nor implied.

For Quick, Clean, Low-Cost Conveying . . . Install FULLER-KINYON PUMPS

Fuller-Kinyon Stationary Pump: Used by ready-mix concrete and asphalt plants for unloading Portland cement and filler dust from hopper-bottom cars. Also in many industrial plants for conveying dry pulverized materials from pulverizers, collector screws and bins to process bins in the plant. Capacities to 300 tons an hour. Conveying material up to 5000 feet; elevations to 500 feet.



Fuller-Kinyon Portable Pump: Commonly used for withdrawing pulverized materials from silos and transporting such materials to packers, cars, ships and barges and, if desired, to process bin throughout the plant. The pump, mounted on wheels, is moved on a track from one point of discharge to another. One unit to convey, elevate and distribute to all points.

Fuller-Kinyon Remote-Control Unloader: For unloading bulk Portland cement from box cars, ships and barges. All of the cement used at Grand Coulee, Shasta and Friant Dams was unloaded and conveyed to storage by Fuller-Kinyon Unloaders. In use by ready-mix concrete and asphalt plants for unloading cement and filler dust from box cars and barges.

P-69

FULLER COMPANY

CATASAUQUA—PENNSYLVANIA

CHICAGO, 3 Marquette Bldg. | WASHINGTON, 5, D.C. Colorado Bldg. | SAN FRANCISCO, 4 Chancery Bldg.

A DESIGN FOR TOMORROWS' AIRPORTS *Today!*

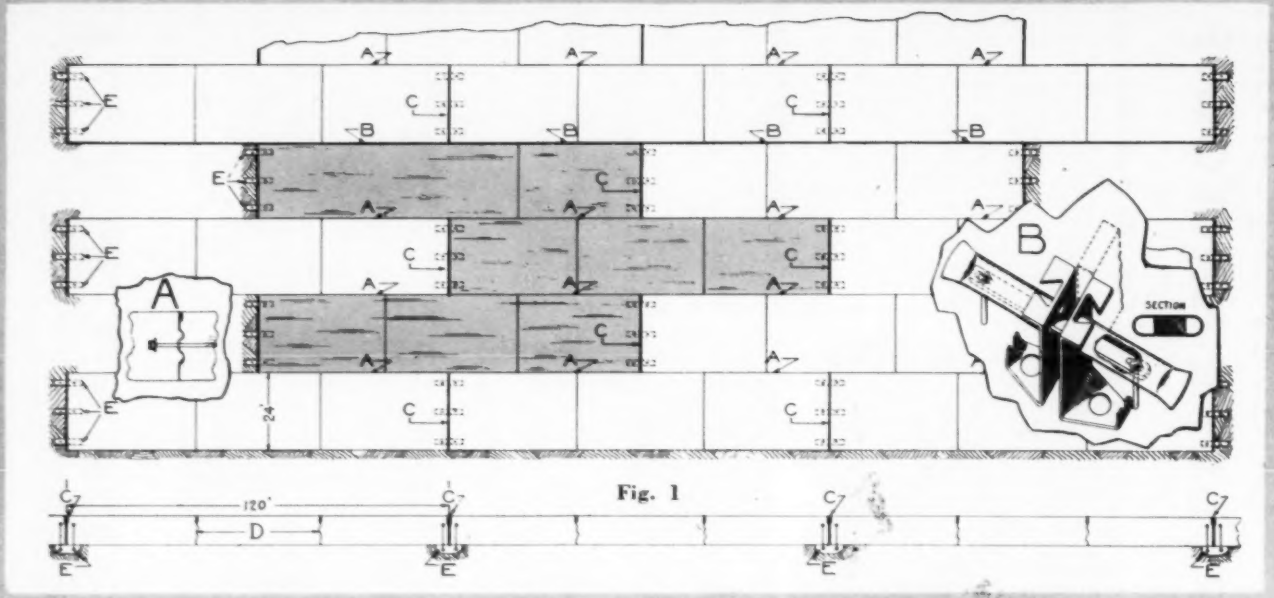


Fig. 1

STAGGERED EXPANSION JOINTS WITH EXPANSION JOINT BEAMS

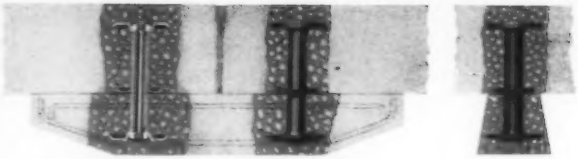


Fig. 2—Helco Universal Tension Dowel and Expansion Joint Beam for Airports (See "E" above)

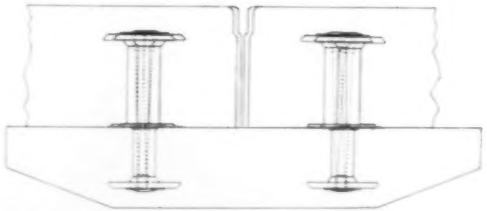


Fig. 3—Note universal action of the Helco Universal Tension Dowel. Allows expansion or contraction in any direction.

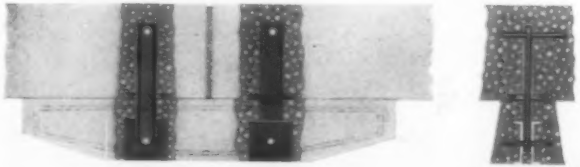


Fig. 4—Helco Tension Dowel and Expansion Joint Beam for Highways

Expansion and contraction of slabs in concrete roads is longitudinal.

Expansion and contraction of slabs in concrete runways in airports hundreds of feet wide is in all directions. This requires certain changes in basic design.

Anticipating these trends, Heltzel Engineers propose a pattern of staggered expansion joints as indicated in Fig. 1. Concrete Expansion Joint Beams with Helco Universal Tension Dowels prevent pumping of joints and provide load transfer elements that permit expansion or contraction in any direction. Indicated as E in Fig. 1.

Slab movement is crosswise and lengthwise of the runways at the longitudinal expansion joint. (See B in Fig. 1.) The Helco Dual Action dowel indicated as B provides a load transfer element and tie that allows for two-way movement.

Complete details and data on load transfer elements, doweling system and expansion control for concrete pavements of any thickness or width are available.

INVESTIGATE

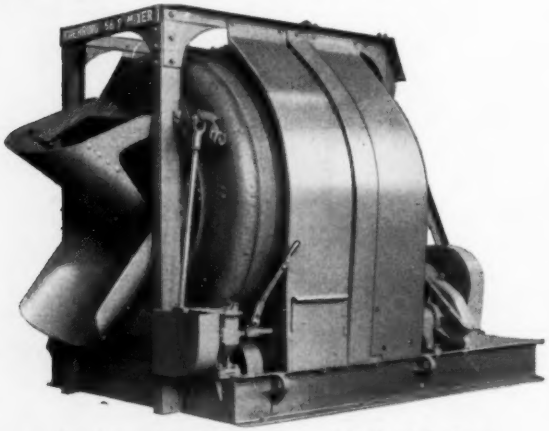
HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO, U.S.A.

KOEHRING

TILTING AND NON-TILTING HEAVY-DUTY CONSTRUCTION MIXERS

Koehring Concentric Zone Tilting Mixer 56-S, 84-S, 112-S

Important progress in the design of Koehring Tilting Construction Mixers during the past years has been made to further the production of better concrete . . . concrete best suited for the large volume dam structures and other similar projects. These improvements, because they are responsible for better concrete, also permit concrete production in a shorter mixing period. Large drum single opening, 15° above horizontal mixing position and 60° below horizontal discharge position are a few of the important progress changes in the design of Koehring Tilting Mixers.



Koehring Non-Tilting Mixer 28-S, 56-S, 84-S

Tilting mixers are built in sizes 2, 3 and 4 cubic yards of mixed concrete and can be used in concrete plants singly or in batteries of 2, 3, 4 or 5 using the concentric zone plan for discharge into one central hopper. Charging is also from one central group of bin and batchers.

Non-Tilting Mixers are available in sizes of 1, 2 or 3 cubic yards per batch. The Koehring Flow-Line discharge chute for minimum of segregation is one of many recent design improvements.

KOEHRING COMPANY - - Milwaukee, Wis.

A NEW MACHINE FOR A NEW ERA

VIBER SLAB

ENGINEERS know that a period of unparalleled construction activity begins when Victory is won. But plans, methods, equipment can't wait for the final gun. Viber Company, pioneer in producing concrete internal vibration equipment, has slanted its research and experimentation to the new specifications of postwar. One such development is the VIBER SLAB, a multiple, high-speed internal concrete vibrating machine specially designed to help build tomorrow's heavy-duty airport runways, aprons, highways.

Tests already performed definitely reveal: (1) VIBER SLAB will vibrate the stiffest concrete on slabs as fast as it can be economically placed on the sub-grade. (2) The battery of "Vibers" will vibrate the ENTIRE DEPTH *(not just the top surface)* of super-thick slabs. (3) Viber's "Viber-ation" in low slump concrete slabs produces strengths not now obtainable with present day equipment, and at a new low cost not now approached.

Expect big things of VIBER SLAB . . . a new machine for a new era!

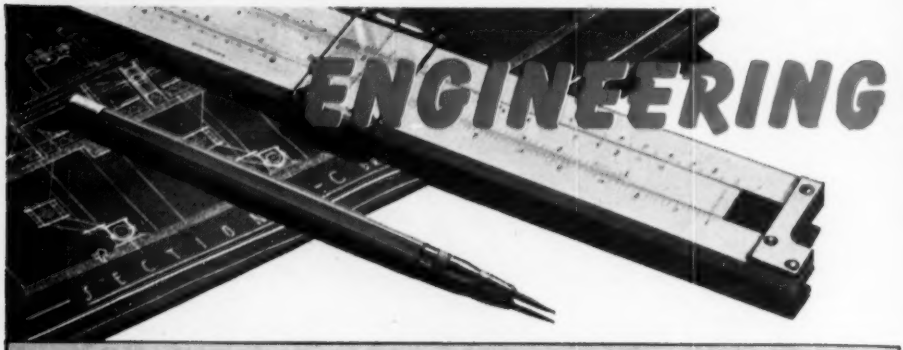
The new SLAB is a development based on the superior performance of famous VIBER vibrators.

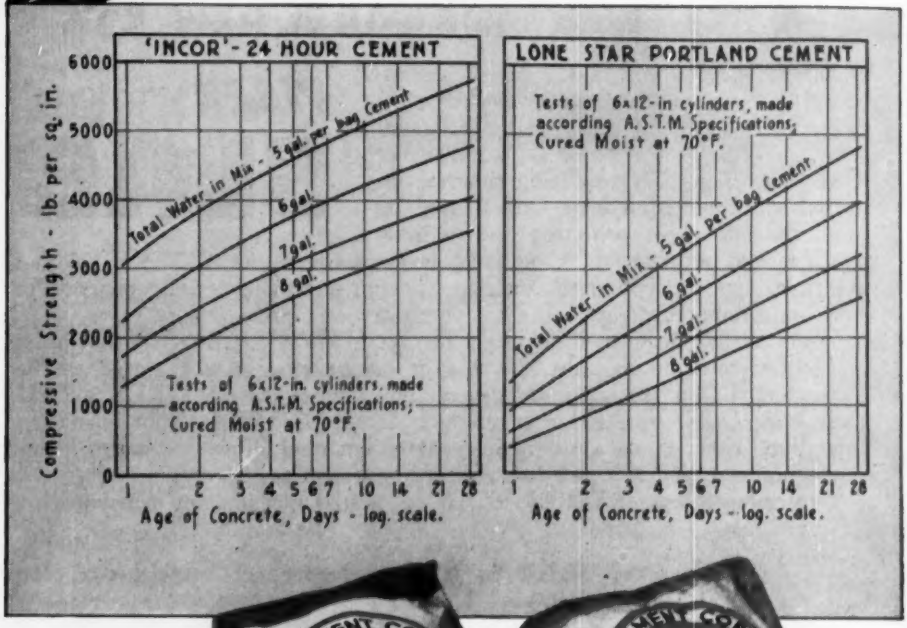
In the field for years, VIBER internal concrete vibrators have been delivering constant satisfactory service under all job conditions. VIBERS speed up the job, providing the highest vibration speed in concrete—over 9500 RPM!—at 110 Volts. Easy to work with, portable, rugged—no wonder VIBER gets the call! VIBER'S simplified design with complete interchangeability of parts including power units (electric, pneumatic or gasoline) further eliminates costly delays and lowers costs. So specify VIBER-ation (not ordinary vibration) and put VIBERS on the job.

VIBER COMPANY

726 South Flower Street, Burbank, California

ORIGINATORS OF INTERNAL CONCRETE VIBRATION




'INCOR'- 24 HOUR CEMENT
Compressive Strength - lb. per sq. in.
6000
5000
4000
3000
2000
1000
0
Total Water in Mix - 5 gal. per bag Cement
6 gal.
7 gal.
8 gal.
Tests of 6x12-in. cylinders, made according A.S.T.M. Specifications; Cured Moist at 70°F.
1 2 3 4 5 6 7 10 14 21 28
Age of Concrete, Days - log. scale.
LONE STAR PORTLAND CEMENT
Tests of 6x12-in cylinders, made according A.S.T.M. Specifications; Cured Moist at 70°F.
Total Water in Mix - 5 gal. per bag Cement
6 gal.
7 gal.
8 gal.
1 2 3 4 5 6 7 10 14 21 28
Age of Concrete, Days - log. scale.

Lone Star Cement Research Laboratories furnish up-to-date performance data, to help Contractors, Engineers and Architects design mixes and plan schedules. Continuous research and rigid manufacturing control assure utmost quality and uniformity.

LONE STAR CEMENT, WITH ITS SUBSIDIARIES, IS ONE OF THE WORLD'S LARGEST

ECONOMICAL SPEED
INTO CONCRETE CONSTRUCTION

FORMS and time are principal cost factors in all concrete work. These costs are governed by the time it takes concrete to attain service or stripping strengths. Advance planning makes it possible to select for each job, or part of the job, the type of Portland cement which produces the lowest-cost schedule.

Between them, Lone Star Cement and 'Incor' 24-Hour Cement cover the entire construction range. **Lone Star Cement** usually shows the lower concreting cost where forms are not tied up waiting for concrete to harden. **'Incor' 24-Hour Cement** shows the lower cost where design adapts itself to rapid form re-use, as well as when heat-protection is required. For equal job speed, 2 to 2½ times more forms are required with Type I cement.

Selecting cement to fit the job produces important economies—cost records show *net* savings up to $1.96 per cu. yd. of concrete in place and ready to use. Field experience since 1927 proves 'Incor'* advantages on—

STRUCTURES: 24-hour form removal . . . 50% less forms. No re-posting . . . ready for immediate use.

HEAVY-DUTY FLOORS: Overnight service . . . superior wear-resistance with only 24-hours curing. For severe exposure, cure 48 hours.

WATERTIGHT CONCRETE: Thorough curing, in 1-2 days, saves 5-7 days. Smoother-working mixes place easier, no separation.

WINTER WORK: Now economical . . . only 1-2 days heat curing . . . saves forms, heating, tarpaulins.

Let job analysis decide—selective concreting assures quality concrete at lower cost.

*Reg. U. S. Pat. Off.

LONE STAR CEMENT CORPORATION

Offices: ALBANY · BIRMINGHAM · BOSTON · CHICAGO · DALLAS · HOUSTON · INDIANAPOLIS · JACKSON, MISS. KANSAS CITY, MO. · NEW ORLEANS · NEW YORK · NORFOLK · PHILADELPHIA · ST. LOUIS · WASHINGTON, D. C.

CEMENT PRODUCERS . . 15 MODERN MILLS . . 25 MILLION BARRELS ANNUAL CAPACITY

Feel the Grip of the HI-BOND Bar!

Take a firm hold on a piece of Inland HI-BOND Reinforcing Bar and note its grip. This is important to you because when HI-BOND bars are placed in concrete they assure a more effective mechanical grip irrespective of the position in which they are cast or the direction in which they are pulled.

The Inland HI-BOND Bar gives the first real improvement in the bonding value of reinforcing bars in more than 30 years. The scientific design of this new Inland HI-BOND concrete reinforcing bar, with its reversed double helical ribs, provides vastly greater anchorage and bonding strength. It provides more efficient transfer of stress at splices. The use of HI-BOND Bars will materially reduce the width of cracks, thereby reducing the possibility of corrosion and preserving the appearance and safety of reinforced concrete members.

A new technical bulletin showing the results of extensive laboratory tests, which will be of interest to all architects, engineers and contractors, is available upon request. Write for your copy.

INLAND STEEL COMPANY

38 S. Dearborn St., Chicago 3, Ill.

Sales Offices: Cincinnati • Detroit • Kansas City • Milwaukee • New York • St. Louis • St. Paul

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Non-Shrink Metallic Aggregate

KEMOX is used integrally and on the surface of horizontal or vertical concrete masonry structures to bond, waterproof, or harden. In action, KEMOX is both chemical and mechanical.

KEMOX, like many famous products became a war casualty and was "shelved" for the duration.

In the field of research we have remained active, and when final Peace comes to America and the World, Kemox will emerge from enforced retirement to assume it's original role to Industry as a dependable medium for solving concrete and masonry problems.

Meantime we continue to buy War Bonds.

CONCRETE MASONRY PRODUCTS COMPANY

220 Denrike Building, Washington, D. C.

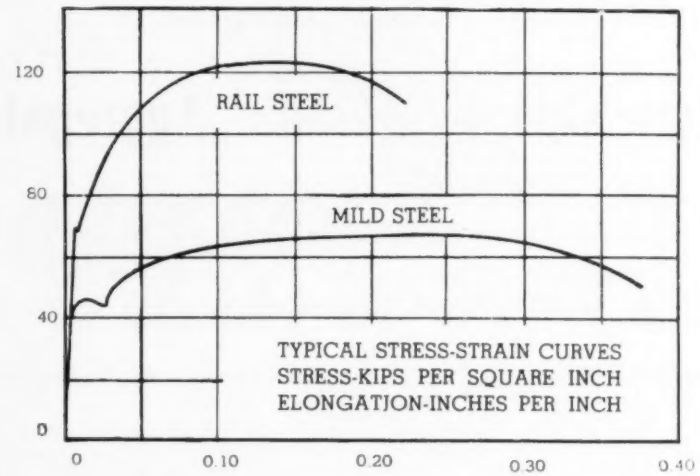
RAIL STEEL
REAL STRENGTH

ADEQUATE !!

- recently at maximum increased working stresses under emergency design regulations.
- in the future for any economical design trend to increased use of material properties.

RAIL STEEL CONCRETE REINFORCEMENT BARS

provide an elastic limit above 50,000 pounds per square inch and 20 tons of tensile strength above the elastic limit for a maximum safety factor and the averages are above these figures.



Rail Steel — is available at once for properly authorized projects.

RAIL STEEL BAR ASSOCIATION

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Franklin, Pennsylvania
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Laclede Steel Company
St. Louis, Missouri
Missouri Rolling Mill Corp.
St. Louis, Missouri
The Pollak Steel Company
Cincinnati, Ohio
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Sweet's Steel Company
Williamsport, Pennsylvania
Texas Steel Company
Fort Worth, Texas
West Virginia Steel & Manufacturing Co.
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● **For the third time** Stearns employees have won the Army-Navy "E" award for excellence in war production.

These awards, in which we take pride and for which we are deeply grateful, have in no way decreased our interest in the field we normally serve.

● **More Stearns Machines** are at work in concrete products plants than those of any other manufacturer . . . **See map** . . . **Today,** our production of these machines almost equals peace-time levels.

● **Of importance to you** is the fact that our research engineers have developed a new and revolutionary method for the mass production of concrete units. This development, now being incorporated into a new, great line of Stearns Block Machines, will be announced at the close of the war.

● **Stearns leads** its field in war production and in the design and manufacture of concrete products plant machinery.

STEARNS MANUFACTURING CO. - ADRIAN, MICH.

GENE OLSEN, PRESIDENT

Pioneers in the manufacture of machinery for the mass production of concrete units by vibration.

Tamp-type machines .. Mixers .. Skip Loaders

RIEHLE

Precision Hydraulic Compression Testing Machines for Testing Concrete

Capacities 120,000, 200,000 and 300,000 Pounds

FOR OVER 70 YEARS Riehle has designed and manufactured hydraulically operated testing machines which have been marketed in every part of the globe.

RIEHLE OPERATING AND CONSTRUCTION FEATURES

Convenient reading level of indicators
Handy speed control hand wheel and load release valve
Rigid and rugged construction
All vital parts enclosed

ALSO AVAILABLE . . . Cement Briquette Tensile Testing Machines, Cap. 1,000 Lbs.

For Testing Gypsum and Gypsum Products . . .
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Write for descriptive literature and quotations.

RIEHLE *Hydraulic Testing Machines*

Riehle Testing Machines Division, American Machine and Metals, Inc.

East Moline, Illinois

"Not Too Hard
Not Too Soft
Just Right"

'JUST RIGHT' to permit the upper portion of a concrete slab to expand. 'JUST RIGHT' in making a permanent seal. Ribbon joint functions 'JUST RIGHT' in many other ways.

Many Engineers are acquainted with 'FLEX-PLANE' joint installers and find there is no substitute. Over a long period of years 'FLEX-PLANE' ribbon joint has stood up almost perfectly.

When you install dummy joints of any kind or type, be sure to use 'FLEX-PLANE' installers.

FLEXIBLE ROAD JOINT MACHINE CO.

WARREN, OHIO, U. S. A.

KALMAN

ABSORPTION-PROCESS

Cross Section of Kalman Floor Topping,
Showing Uniform Distribution of Aggregate and Density

THE KALMAN FLOOR—A GRANOLITHIC CEMENT FINISH FLOOR TOPPING. Laid by the Kalman Floor Company using the KALMAN ABSORPTION PROCESS.

The Kalman Floor Company has for 28 years pioneered and specialized in the development and installation of Granolithic Cement Finish Floor Topping and has during this period maintained continuous leadership in the field through scientific research and experiments.

KALMAN FLOOR COMPANY
INCORPORATED
110 EAST 42ND ST. NEW YORK 17, N. Y.

FLOOR

CEMENT-FINISH

The Kalman Absorption Process

A practical method of installing on a commercial basis a theoretically correct low-water concrete topping. This method, combined with the skill and knowledge of the men doing the work and subjecting all materials to Laboratory test for analysis, gradation and soundness, plus a constantly growing skilled personnel and improvement in equipment has consistently produced a uniformly hard wear-resisting floor, free from disintegration and dusting and of a maximum density, evenness of texture and unexcelled durability.

Adaptability

Over 200,000,000 Sq. Ft. of Kalman Floors have been installed throughout the United States in practically every type of Industrial, Warehousing, Institutional, Commercial and School buildings and is readily adaptable. (For further information, see Sweet's Architectural Catalog. A. I. A. File No. 4 i 3.)

OFFICES:

NEW YORK	CHICAGO	BOSTON
LOS ANGELES	PHILADELPHIA	CHARLOTTE
SEATTLE	CLEVELAND	DAYTON

Anticipating Tomorrow's

1 CONCRETE RE-MIXED ON THE SUB-GRADE by compacting spreader screw:

Comparative tests by highway engineers of various States have proved conclusively that the Jaeger method of screw-spreading concrete produces a more uniform, denser and, therefore, longer wearing slab.

By its thoro and positive re-mixing and inter-mixing of piles dumped on the subgrade, both the segregation of coarse aggregates in the batch and the variations between different paver batches are eliminated; badly placed batches are redistributed to leave a uniform spread of material ahead of the Finishing Machine, with material placed so solidly against the road base and side forms as to eliminate the honeycomb problem and the entire mass compacted to weight and density approaching that of vibrated concrete.

As one prominent engineer states: "It has been demonstrated that the quality of concrete can be improved and at the same time cost of production to the contractor can be reduced."

It seems logical to expect that re-mixing on the subgrade will be specified where it is desired to insure highest strength, longest life pavements.

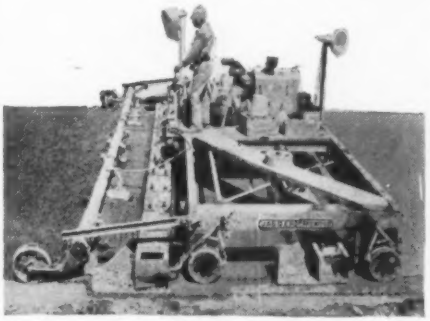
THE JAEGER MACHINE COMPANY, COLUMBUS, OHIO

Paving Specifications . . .

2 VIBRATION ON THE FINISHER, not on the concrete spreader:

Although Jaeger can furnish a vibratory attachment for use on Spreaders, the recommended Jaeger method of vibration *on the Finisher* has proved superior for any true vibratory mix. On an efficiently run job, only the Finisher has time to go back for more than one vibratory pass, as often needed. Also, it is the machine which always finishes to form level, thus insuring an over-all vibrated surface.

It has been demonstrated that specifications for true vibratory mixtures can best be met by use of the Jaeger Vibratory Finisher with "bullnose" screed giving DEEP INTERNAL VIBRATION. For less difficult mixtures use of a vibratory tube, *on the Finisher,* is also satisfactory.

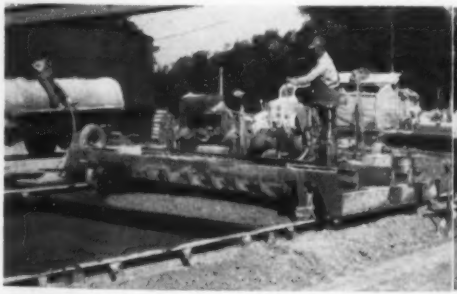


3 FAST, MECHANIZED PLACEMENT and FINISHING of quick-drying air-entraining cements:

The Spreader-Finisher "team", originated by Jaeger, which made it possible to handle stiff, vibratory concrete at the dual-drum paver pace, also equips road builders to handle quick-drying air-entraining cements.

Under hot, windy or dry air conditions, the Jaeger "team" provides the spreading and finishing capacity needed to keep close behind big pavers and complete the job before drying hinders a satisfactory finish.* Also, an exclusive feature of the Jaeger Finisher is that screed speeds can be independent of traction, permitting the use of fast screed speeds which has been found necessary to prevent tearing when the surface is sticky.†

* † Discussions of these two problems have appeared in previous issues of the Journal of the American Concrete Institute.

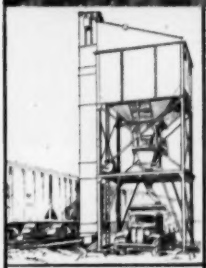


Manufacturers of Concrete and Bituminous Spreading and Finishing Equipment, Form Tampers, Road Forms, Concrete and Bituminous Mixers, Truck Mixers, Pumps, Air Compressors, Hoists, Yard Cranes.

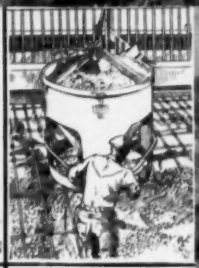
BLAW-KNOX DIVISION of Blaw-Knox Company
Farmers Bank Bldg. • Pittsburgh, Pa.—New York, Chicago, Birmingham, Philadelphia, Washington

USE THIS UP-TO-DATE PAVING METHOD

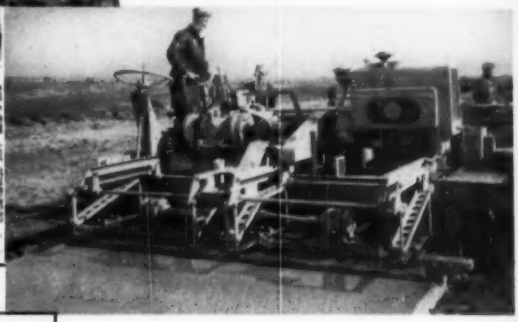
Dry, harsh, concrete paving mix being handled by Blaw-Knox Transverse-Blade Automatic Type Concrete Paving Spreader equipped with vibratory attachment. Concrete tested ½ to ¾ inch slump. Contractor's production in spite of difficult concrete was in excess of 400 lineal ft. of 12 ft. wide slab 9" thick per hour. Spreader-Vibrator is one man operated. Vibration increased strength of concrete by 25 per cent.

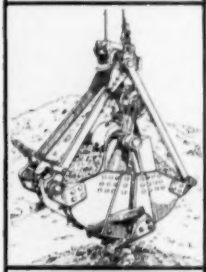


Bulk Cement Plants

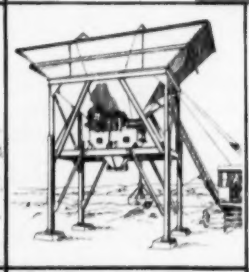


Concrete Buckets



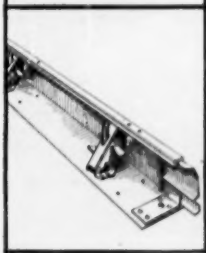


Clamshell Buckets

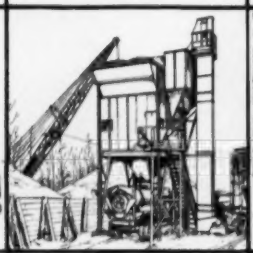


Aggregate Batching Plants

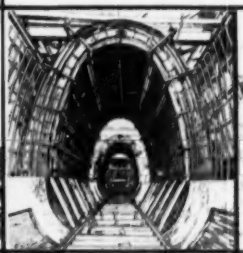
View behind Blaw-Knox Spreader-Vibrator shown in upper photograph. Concrete has been spread to required elevation and simultaneously compacted by vibratory attachment. Note uniformly smooth surface behind vibrator. Blaw-Knox Finishing Machine worked closely behind Spreader-Vibrator and kept pace easily. Cores drilled from completed pavement showed no honeycomb at bottom of slab or at joints and no excess mortar at surface of pavement.



Paving Forms for Roads and Airports



Truck Mixer Loading Plants

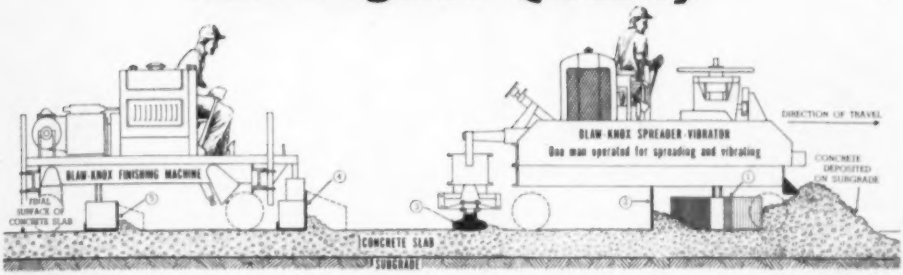


Steel Forms for General Concrete Construction

Steel Street Forms

• BLAW-KNOX •

for Greater Production, Lower Cost and Higher Quality



1 Automatic Transverse Spreading Blade spreads concrete transversely and at the same time pushes excess concrete ahead of machine; adjustable for spreading height.

2 Strike-off shapes concrete to required height and crown allowing slight excess for compaction by vibrator; strike-off is hydraulically adjustable for elevation.

3 Vibratory attachment compacts concrete simultaneously with spreading operation; vibrator is spring suspended and does not rest on side forms. All vibratory effect is transmitted directly to the concrete. Vibrator is controlled by spreader operator and leaves slight excess of concrete for finishing machine.

4 Finishing Machine front screed strikes off excess of concrete to exact grade and crown. Finisher has easy and rapid operation; follows close behind Spreader-Vibrator.

5 Rear screed of Finishing Machine performs final finishing and smoothing operation.

The method of paving construction illustrated has been proved on hundreds of miles of concrete paving construction for roads and airports.

The dry and harsh concrete mixes frequently specified by engineers for modern pavements can be spread, compacted and surfaced most rapidly and efficiently by the combination of the Blaw-Knox Transverse-Blade Type Automatic Concrete Paving Spreader equipped with vibratory attachment and the modern Blaw-Knox Finishing Machine.

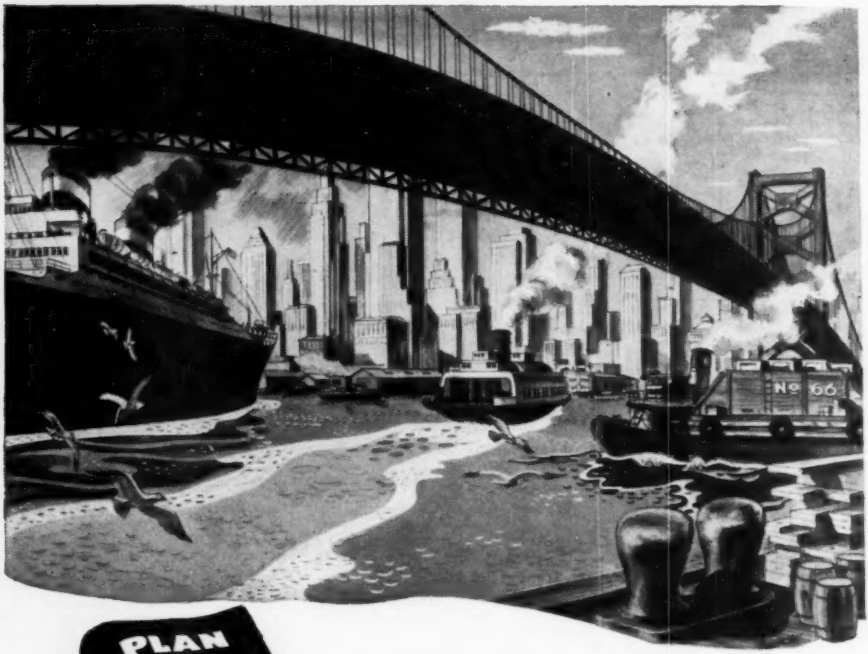
The Spreader-Vibrator spreads the concrete to the required depth and at the same time compacts the concrete by vibration. The Finishing Machine follows close on the heels of the Spreader-Vibrator and does a quick and easy surfacing job. The Blaw-Knox Spreader-Vibrator teamed with the Blaw-Knox Finishing Machine handles the output of two 34-E dual drum paving mixers.

Difficult concrete is easily handled on a production basis by this up-to-date paving method and the contractor gains — in greater yardage, lower construction cost, minimum of manual operations and higher quality paving.

The Blaw-Knox Finishing Machine can also be equipped with a vibratory attachment. However, experience has shown that the paving vibrator mounted on the spreader provides better compaction, more practical operating procedure, and maximum production of paving slab. The Spreader-Vibrator always remains with the paving mixer and does not have to move back to aid in correction of high or low areas.

Blaw-Knox Spreaders and Finishers including vibratory attachments are available in standard sizes as follows: 10-15 ft. adjustable width, 20-25 ft. adjustable width.

Your Nearest Blaw-Knox Distributor Will Promptly and Efficiently Handle Your Inquiries for Construction Equipment.



...FOR THE WORK TO BE DONE TOMORROW

Now is the time when industry should turn to architects, engineers and general contractors for preparing practical, well-designed plans for the new buildings, piers, bridges, harbors, dams and other important post-war developments.

Only by planning today will we be ready with the jobs we have promised our fighting men. Plans cannot be made overnight; that's why it is so essential for us to do our share of the work now. Let's remember immediate jobs are what our men want when they come home. To fulfill the promise we have made is the only way of saying, "Thanks, for what you have done!"

The Raymond organization stands ready to serve industry with practical, economical ways and means gained through the successful completion of over 10,000 contracts of every size and type, and under every sort of condition. A Raymond representative is available for an immediate conference.

RAYMOND CONCRETE PILE CO.

Branch Offices in Principal Cities

140 CEDAR ST. • NEW YORK 6, N. Y.

SCOPE OF RAYMOND'S ACTIVITIES *includes every recognized type of pile foundation—concrete, composite, precast, steel, pipe and wood. Also caissons, construction involving shore protection, ship building facilities, harbor and river improvements and borings for soil investigation.*

Restore the Old-- with Build the New--

INTRUSION-PREPAKT

INTRUSION: Pressure-filling of spaces within masonry and concrete structures, foundations, and riprap with cement-base binding paste, to solidify the mass.

PREPAKT: Concrete made by packing forms with coarse aggregate and then solidifying by intrusion.

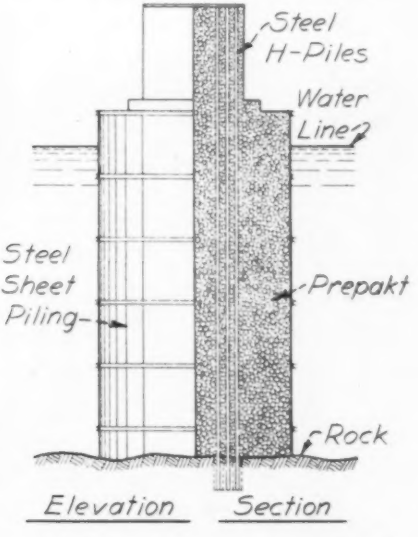
Intrusion and Prepakt are used separately or together to restore old masonry and concrete structures and as special construction in difficult locations, underwater work, and wherever unusually low shrinkage is required.

Structures in which Intrusion and Prepakt have been used successfully include

PIERS	DAMS
FOOTINGS	ROCK FILLS
ARCHES	BREAKWATERS
WALLS	TUNNELS

Further information is given on this and the three following pages.

Restoration of bridge pier above and below water. Damaged stones replaced with Prepakt, interior of pier solidified by Intrusion. No caisson necessary: sheet-piling shield across nose of pier to deflect current for divers.



Bridge pier to be built in 120 ft. of water. Guide frame holds sheet piling and guide tubes for steel H-piles which are set in anchor holes in rock. Forms filled with coarse aggregate which is then intruded to form Prepakt concrete, without unwatering.

RESTORATION BY INTRUSION

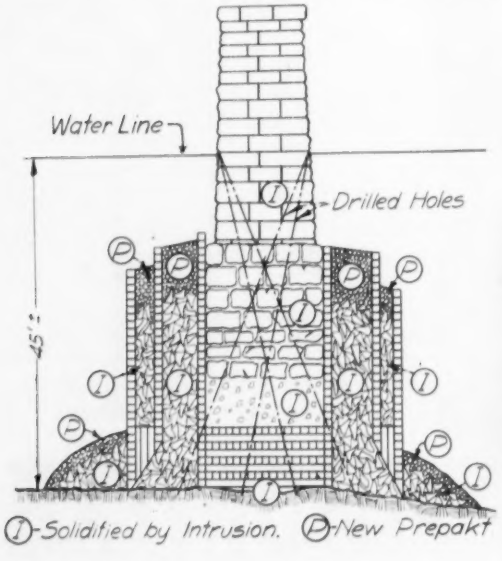
AN EXAMPLE

CONDITIONS: Ten stone masonry piers and abutments of railway bridge built in 1870. Each pier built on concrete base in timber box, within timber caisson; caisson filled with riprap to stabilize pier (see diagram of typical pier, below). At time of restoration the masonry was weathered and spalled, and mortar joints were in generally bad condition. Water was 10 to 50 feet deep, current 8 to 12 miles per hour. No interruption to traffic could be permitted.

METHOD OF INTRUSION: Holes were drilled from just above water line to provide all parts with inlets at intervals. Intrusion mortar was pumped in under pressure to solidify stone masonry, concrete-timber base, rock foundation, and riprap. Riprap was capped with Prepakt, the aggregate and intrusion pipes for which were placed by aid of divers. Job required about 30,000 lineal feet of drilling, about 1,250 cubic yards of intrusion mortar, and about 250 cubic yards of Prepakt. No coffer dams were necessary—only a line of steel sheet piling across the nose of the piers to slow the current for divers.

QUALITY

Thoroughness of intrusion is evidenced by appearance of mortar at adjacent openings and by test cores cut from structure. Compression tests on cores have repeatedly demonstrated strength of solidified structure at least equal to that of original material. Quality of intrusion mortar is necessarily varied to suit the particular conditions, but in any case the water-cement ratio is relatively low as compared with straight portland-cement grout, and the strength is ample. Laboratory tests on intrusion mortars show compressive strengths up to 10,000 psi. at 3 months, also little bleeding and low shrinkage. Test data available on request.



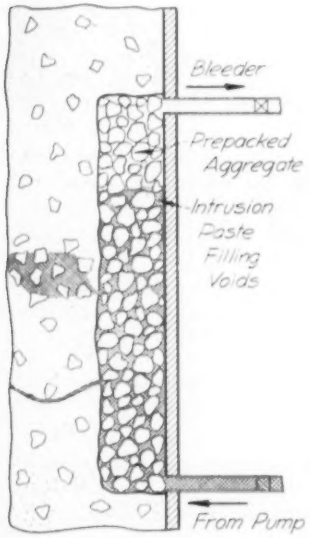
I-Solidified by Intrusion. P-New Prepakt

RESTORATION WITH PREPAKT

Dense—Strong—Durable
Tight Bond—Low Shrinkage

Prepakt concrete consists of closely spaced coarse aggregate embedded under pressure in a matrix of dense, non-bleeding, low-shrinkage mortar.

Disintegrated parts of concrete and masonry structures are cut out and replaced with Prepakt. Pressure filling assures tight contact and unit action with existing structure; also the intrusion paste under pressure penetrates cracks and voids in the adjacent masonry, thus cementing and stabilizing the mass. The close contact between aggregate pieces and the special characteristics of the intrusion paste assure low shrinkage and therefore continued bond and unit action of Prepakt with structure.



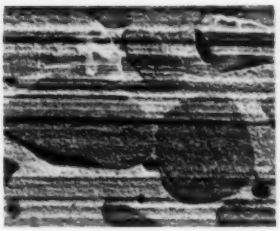
Typical arrangement for restoration. Overhead and underwater placing are equally effective.

Quality of Prepakt is demonstrated by experience in many structures, exposure to weathering, and tests for strength and bond to old concrete. Strength is varied to meet requirements; normally compressive strength is in excess of 3200 psi. at 28 da., 4,000 psi. at 3 mo., and 5,000 psi. at 1 year. Typical laboratory tests show shrinkage of Prepakt about half that of ordinary concrete, and cracking practically non-existent. Test data available on request.

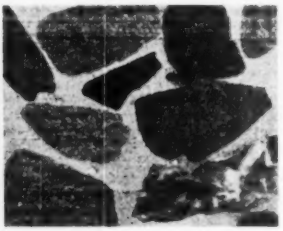
Railway tunnel strengthened without impairment of clearance by means of curved steel beams set in slots and encased in Prepakt. Intrusion paste penetrating original structure often appeared several feet away.

NEW CONSTRUCTION WITH PREPAKT

Uniform Placing
Low Shrinkage
Density
Strength
Durability

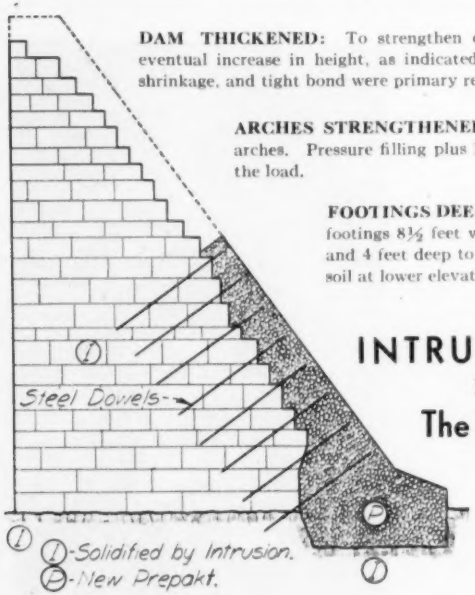


Ordinary concrete — often water pockets under aggregate.



PREPAKT cast under water— solid throughout.

Locations and conditions difficult for placing ordinary concrete are particularly suited to Prepakt. Placing and inspection of coarse aggregate before intrusion assures complete filling of forms. Limited size range prevents segregation even when aggregate is placed horizontally or overhead. In underwater construction the intrusion mortar displaces water from below; tests demonstrate that the advancing surface does not mix with the water as does straight portland-cement grout. Dry-dock floors, sea walls, piers, or structures under water, where concrete is subject to wash and segregation, can be built rapidly and economically with Prepakt. Following are examples of actual jobs where Prepakt met unusual conditions requiring tight and continued contact with surroundings in addition to usual requirements of density, strength, and durability.



DAM THICKENED: To strengthen dam, stop leakage (by intrusion), and permit eventual increase in height, as indicated by sketch below. Low heat generation, low shrinkage, and tight bond were primary reasons for adopting Prepakt.

ARCHES STRENGTHENED: Prepakt arches supporting old stone masonry arches. Pressure filling plus low shrinkage enabled Prepakt portion to share the load.

FOOTINGS DEEPENED AND EXTENDED: Under bridge pier footings 8½ feet wide were built Prepakt footings 15½ feet wide and 4 feet deep to spread the load and transfer pressure to firmer soil at lower elevation.

For Restoration

INTRUSION-PREPAKT, Inc.

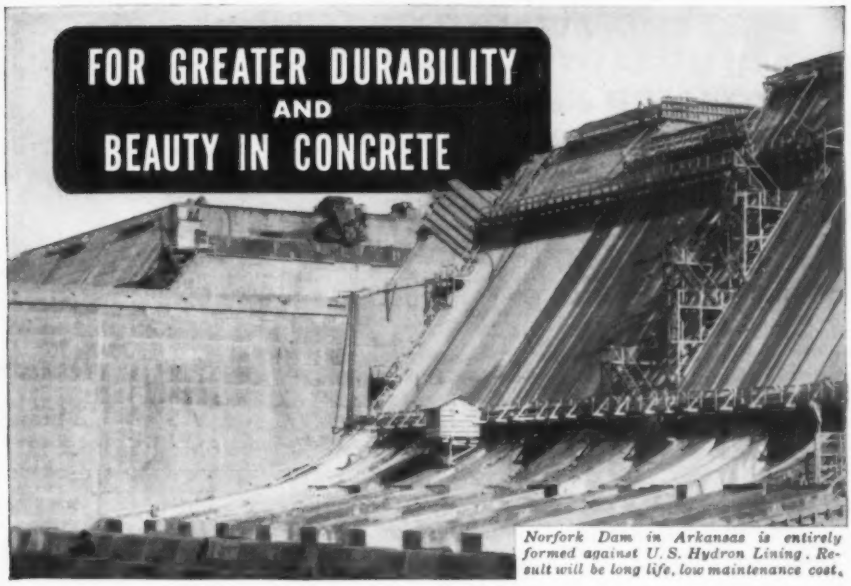
For New Construction

The PREPAKT CONCRETE Co.

Union Commerce Bldg., Cleveland 14
309 W. Jackson Blvd., Chicago 6
429 Union Station, Toronto, Ont., Can.

Methods and Materials Patented

See Also Preceding Three Pages



Norfork Dam in Arkansas is entirely formed against U. S. Hydron Lining. Result will be long life, low maintenance cost.

HYDRON*

*Trade Mark

THE NEW ABSORPTIVE FORM LINING

U. S. Hydron produces finer looking, longer lasting concrete by removing from the surface the water and air bubbles found often in mixes with a low water-to-cement ratio.

Moreover, this new absorptive lining actually increases resistance to surface abrasion and freeze-thaw action. Hydron uses no critical materials and is easy to ship, to store and to apply.

Tests show that the case-hardening effect produced by Hydron lining is at least one inch deep, and there is a gradual change in water-to-cement ratio from the surface into the bulk concrete.

HYDRON EASY TO USE

Thin and light—only 0.08 in. thick; weighs 0.22 lb. per sq. ft.

Can be cut or trimmed with knife or scissors.

Mounted to forms by use of rapid fire staple guns.

No special forms needed; dimensional adjustments unnecessary.

Flexible—fits curved or arched surfaces.

Strips cleanly—no tedious finishing.

Send for your copy of informative booklet on Hydron

UNITED STATES RUBBER COMPANY

Mechanical Goods Division • Rockefeller Center • New York • In Canada: Dominion Rubber Company, Ltd.

AUTHORITATIVE TESTS
show
POZZOLITH gives VASTLY GREATER DURABILITY with INCREASED STRENGTH

In an exhaustive investigation employing a range of cements typical of all U. S. Cements — (Full reports available for examination.)

1. Pozzolith increased DURABILITY —

SEA WATER — after 142 Cycles of Outdoor Freezing and Thawing

Nominal Cement Factor—Sacks		
6	Plain Concrete — of 39 beams tested	15 survived
6	Pozzolith " — " 77 " "	76 survived

FRESH WATER — In Laboratory — Freezing and Thawing

	After Average No. Cycles	Average Durability Factor	Average Change in Dynamic E
4½ Plain Concrete	49	17	—31
4½ Pozzolith "	139	63	—21
6 Plain "	50	20	—32
6 Pozzolith "	176	80	—10

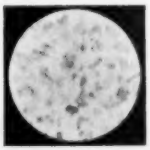
2. Pozzolith increased STRENGTH —	6 Sack Mixes	4½ Sack Mixes
3-day Compressive	50%	70%
28-day "	12%	25%
3. Pozzolith cut BLEEDING	55%	60%
4. Pozzolith cut WATER-CEMENT Ratio	14%	15%

POZZOLITH . . . What It Is and How It Works —

Pozzolith in powder form is a combination of the Cement Dispersing Agent calcium-lignosulfonate (a colloidal electrolyte), the accelerator — calcium chloride, and the pozzuolana — low carbon fly ash. In accordance with the classification of admixtures given by Committee 212 in "Admixtures for Concrete", J.A.C.I. Vol. 16 No. 2, Nov. 1944, Pozzolith belongs primarily in group 8b, and secondarily in groups 1 and 2.

HOW IT WORKS

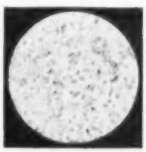
Cement particles in their normal state in water tend to flocculate. Water never reaches some particles and others are only partly hydrated. This reduces the effectiveness of the cement, entraps water within the clumps, requires an excess of water for placement and often results in bleeding and segregation.



UNDISPERSED

As the colloidal lignosulfonate anion is absorbed by the cement particles endowing them with negative electric charges, dispersion results, thus exposing all the cement particles to the vital hydrating action and consequently assuring increased efficiency.

This makes the cement usable to far greater efficiency since all the water is made available for lubrication of the mix, and a much greater surface area of the cement is exposed for hydration.



DISPERSED

SUMMARY OF REPORTS OF AUTHORITATIVE TESTS OF POZZOLITH

SCOPE

More than 800 mixes and more than 1800 concrete specimens were made and tested in the course of the investigation, which was made by a top independent, non-commercial testing authority.

Cements used in the investigation include 8 Type I, 3 Type II, 1 Type III, and 1 Type IV from 12 mills located in 10 states. Chemical analyses and the conventional physical tests, in accordance with Federal Specifications SS-C-158a, were made of the cements.

The Pozzolith used in the investigation included 11 different lots obtained over a period of 60 days by representatives of the Authority sponsoring these tests, and were taken principally from construction jobs in various localities in the United States. Thirty-one individual Pozzolith samples were taken from 31 separate packages. Results of chemical analyses made on the 31 individual samples showed the standard deviation for all tests for the dispersing component to be .8%; test method based on duplicate determinations to be .6%.

RESULTS

Durability. Treat Island, Maine, outdoor exposure durability tests (combining freezing and thawing and sea water exposure) on 6 x 6 x 48 inch beams were made with the nominal 6-sack (3½" slump) plain mixes and 6-sack (3½" and 6" slumps) Pozzolith mixes using all cements. After 142 cycles of freezing and thawing and salt-water wetting and drying, 24 or 62% of the original 39 plain concrete specimens had completely disintegrated, whereas 75 or 97% of the original 77 Pozzolith specimens were better in dynamic E than they were before exposure.

Durability. Laboratory freezing and thawing durability tests on 3 x 3½ x 15 inch beams were made on nominal 6-sack (3½" slump) concrete, using all cements with and without Pozzolith. These freezing and thawing tests were discontinued when any specimen showed a loss in its original dynamic modulus (E) of 30%. After 70 or less cycles of freezing and thawing, 23 of the original 26 plain concrete specimens were discontinued, whereas 1 of the 26 Pozzolith specimens was discontinued. After 135 or less cycles of freezing and thawing, all plain concrete specimens had lost 30% of their initial dynamic modulus (E), whereas only 4 of the original 26 Pozzolith specimens had lost 30% of their dynamic modulus (E). After 200 cycles of freezing and thawing, 14 Pozzolith specimens had a durability factor* of 100 and 3 other Pozzolith specimens had durability factors of 95, 95 and 75.

Durability. Laboratory freezing and thawing durability tests on 3 x 3½ x 15 inch beams were made on nominal 4½-sack (2" slump) concrete, using all cements with and without Pozzolith. These freezing and thawing tests were discontinued when any specimen showed a loss in its original dynamic modulus (E) of 30%. After 91 or less cycles of freezing and thawing, 25 of the original 26 plain concrete specimens were discontinued, whereas 7 of the

*Durability factors were determined in accordance with method described by Stanton Walker: Freezing and Thawing Tests of Concrete made with Different Aggregates; J.A.C.I. June, 1944 — Vol. 40, p. 573, except that limits of minus 30%E and 200 cycles of freezing and thawing form the basis for calculation.

26 Pozzolith specimens were discontinued. After 200 cycles of freezing and thawing, 6 Pozzolith specimens had durability factors of 100, and 5 other Pozzolith specimens had durability factors of 95, 95, 95, 85, and 80.

Strengths. The nominal 6-sack (3½" slump) Pozzolith mixes with all cements had an average increase in compressive and flexural strengths at 3 days of 54% and 25% respectively over the corresponding plain mixes. The Pozzolith mixes at 28 days had average increases in compressive and flexural strengths of 12% and 3% respectively, over the corresponding plain mixes. The nominal 6-sack (6" slump) Pozzolith mixes with all cements had an average increase in compressive strengths at 3 and 28 days of 57% and 13% respectively over the corresponding plain mixes. The nominal 4½-sack (2" slump) Pozzolith mixes with all cements had an average increase in compressive and flexural strengths at 3 days of 71% and 35% respectively over the corresponding plain mixes. The Pozzolith mixes at 28 days had an increase in compressive and flexural strengths of 25% and 6% respectively over the corresponding plain mixes. Inspection of the type of failure in some of the specimens tested in flexure indicated that the coarse aggregate containing a relatively high percentage of mechanically weak pieces of chert and sandstone had adversely affected the later age strengths, with the result that the full mortar strength may not have manifested itself in the 28 day results due to coarse aggregate failure.

Bleeding. Bleeding tests were made on the nominal 6-sack (3½" slump) and 4½-sack (2" slump) concretes with all cements both plain and with Pozzolith added. In the 6 sack mixes the decrease in bleeding of the Pozzolith mixes compared with corresponding plain mixes ranged from 30% to 85% and the average decrease in bleeding for all Pozzolith mixes versus all plain mixes was 55%. In the 4½ sack mixes (2" slump) the decrease in bleeding of the Pozzolith mixes compared with corresponding plain mixes ranged from 25% to 80% and the average decrease in bleeding for all Pozzolith mixes versus all plain mixes was 60%. All Pozzolith mixes showed less bleeding than the corresponding plain mixes.

Water-Cement Ratio Reduction. The nominal 6-sack (3½" slump) mixes made for compressive and flexural strength and laboratory freezing and thawing specimens with all cements had an average water reduction of 15.0% due to the presence of Pozzolith. The nominal 6-sack (6" slump) mixes had an average water reduction of 14.4% due to Pozzolith. The nominal 4½-sack (2" slump) mixes had an average water reduction of 15.4% due to the presence of Pozzolith. The average water requirements for all Pozzolith mixes reported in this investigation ranged from 13.0% to 15.5% less than the average for the corresponding plain mixes of the same consistency.

Over 5,000,000 cubic yards of Pozzolith Concrete used in many of the Nation's leading Structures since 1933 provide complete practical confirmation of the advantages of Cement Dispersion.

Center Strips of Pozzolith Concrete after 5 years service for Bureau of Transportation, City of New York.

Pozzolith Concrete Deck on East Ramp of Main Street Bridge, Cleveland, Ohio.

AUTOMATIC DISPENSERS INSURE ACCURACY IN HANDLING POZZOLITH IN BATCHING PLANTS

Representative Ready Mixed Concrete producers in the United States and Canada employ Pozzolith, Cement Dispersion to produce more durable concrete. Ready Mixed Concrete plants are now being equipped with Master Builders Automatic Pozzolith Dispensers which insure accuracy and minimize handling. The automatic introduction of Pozzolith is rapid and simple. The control dial is set for the pounds or bags of cement in the batch, an electric push button is touched, causing the exact amount of Pozzolith to be delivered into the mix.

Further details concerning the Automatic Pozzolith Dispenser will be supplied on request.

Dual Mixed Concrete & Materials Limited

TORONTO, ONTARIO

July 27, 1944.

The Master Builders Company,
7016 Euclid Avenue,
Cleveland 3, Ohio.

Gentlemen:-

May we say, at this time, that all the advantages indicated have been more than realized in the quality of our concrete, and we are very much pleased with the method of handling, which you have installed, and with the very apparent benefits resulting from its use.

Yours very truly,

DUAL MIXED CONCRETE & MATERIALS LIMITED

Westwood

HDW:AH Vice-President & General Manager

THE MASTER BUILDERS COMPANY

CLEVELAND, OHIO **TORONTO, ONTARIO**

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

Its Contributions to Technical Progress

are in 40 annual volumes (the 41st now appearing periodically in the current ACI JOURNAL) and in many separate publications. . . A few of these are listed below—others on pages 424 and 432.

Recent ACI Standards

Building Regulations for Reinforced Concrete (ACI 318-41)

64 pages in covers: 50 cents per copy. (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

28 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

4 page leaflet: 25 cents per copy

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

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Specification for Cast Stone (ACI 704-44)

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Specifications for Concrete Pavements and Bases (ACI 617-44)

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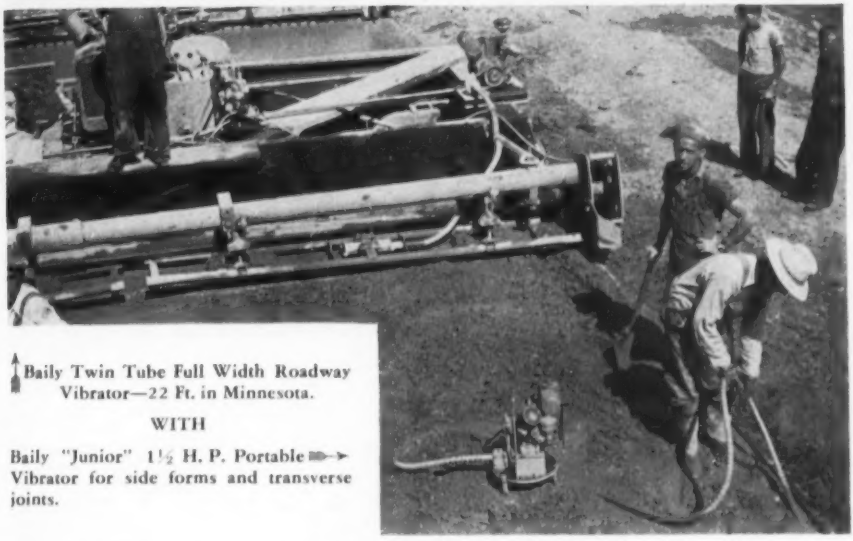
AMERICAN CONCRETE INSTITUTE

742 New Center Building DETROIT 2, MICHIGAN

SAVE $800 A MILE WITH BAILY VIBRATORS



Baily Twin Tube Full Width Roadway Vibrator—22 Ft. in Minnesota.

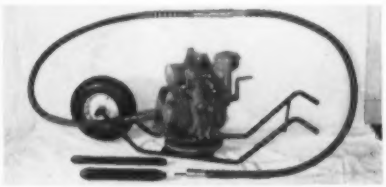
WITH

Baily "Junior" 1½ H. P. Portable Vibrator for side forms and transverse joints.

Yes, by utilizing BAILY VIBRATION in the construction of a 24-foot concrete roadway you can save approximately $800 a mile—and get a better roadway.

Vibration produces a stronger, denser slab with a worth while saving in cement—the most expensive ingredient. Or look at it this way—more miles of road for the same money thereby providing more total employment of labor, a MUST when peace comes.

Baily produced the first vibrating screed for highway construction . . . Baily produced one of the first portable internal vibrators for structural concrete . . . and Baily has continued ever since to pioneer in various types of apparatus for concrete vibration.



Baily 3 H. P. portable vibrator, with wheelbarrow (steel wheels for the duration)

BAILY PORTABLE VIBRATORS for structural concrete. They include gasoline powered flexible shaft units in several sizes, at vibrating speeds from 3600 R.P.M. to 9200 R.P.M.

BAILY ROADWAY VIBRATORS full width types. Surface type vibrating screeds, for attachment to finishers, also for hand-propulsion with power plant carried on the screed.

SOON: Electric and Pneumatic vibrators in modernized design. Baily produced the first commercial compressed air vibrator with motor enclosed in the vibrator head; a patented construction and well known as a product of the principal U. S. A. manufacturer of pneumatic tools.

BAILY VIBRATOR CO. 1526 WOOD ST., PHILA. 2, PA.
DEALERS IN PRINCIPAL CITIES

Yours for the asking

RICHMOND'S
Form-Ty Engineering Guide

Contains product information with helpful charts and tables providing data for the design of form work.

SEE OUR CATALOG IN SWEET'S

• Established 1911 •
RICHMOND SCREW ANCHOR COMPANY, INC.
816 LIBERTY AVENUE • BROOKLYN 8, NEW YORK

VACUUM CONCRETE

(Concrete from which 5 to 8 gals. of water per cubic yard have been removed by controlled Vacuum suction after the concrete has been placed in the forms.)

QUALITY

Water-Cement ratio reduced 1 to 2 gals. per sack of cement.
Doubled 3-day strength; 50% gain at 28 days.
Less Shrinkage—less cracking.
Greater resistance to Wear, and Freezing and Thawing.

SPEED

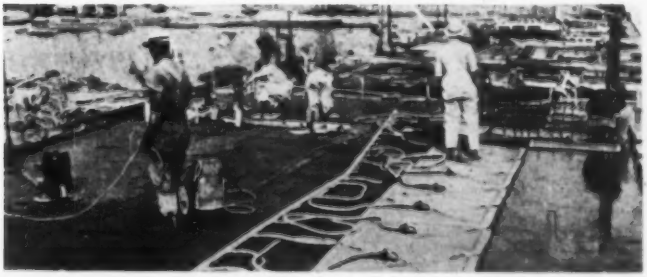
Dewatering permits control of finishing overtime, making possible larger daily pours and late starts to utilize otherwise-wasted part-days.
Removal of free water permits safe concreting at lower Winter temperatures.
Stripping strength at 3 days makes it easy to hold mid-Summer speed all Winter.

ECONOMY

SAVE: Placing:—$25. to $50. per day.
Mix: ⅔¢ to 2¢ per sq. ft.
Finishing: 1¢ to 2¢ per sq. ft.
Dust-Coat or Drier (not needed)
¼¢ to ⅓¢ per sq. ft.
Plus:
Fewer forms—simpler, cheaper Winter protection—earlier completion, etc.
Nothing to buy, ship or store—VACUUM processing equipment is rented.

PROOF

Over 50,000,000 sq. ft. of concrete surface VACUUM-processed for various U. S. Government Agencies including such structures as Shasta Dam (Spillway); Fontana Dam (Spillway Tunnel inverts); U. S. Navy Dry Docks (walls); Concrete Ships.
75% of VACUUM contracts have been with contractors who have used Process on two or more projects.



VACUUM CONCRETE, INC.

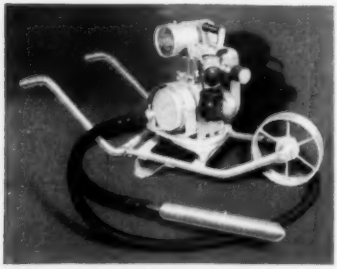
4210 Sansom Street Tel. BARing 4545 Philadelphia 4, Pa.

★ JACKSON ★

Post-war highway construction jobs and airport landing field projects will be of major importance to concrete construction contractors. As pioneers in the field of vibratory concrete equipment, JACKSON can furnish the exact type of equipment for your immediate concrete applications. The complete line of JACKSON vibrators has proved its superiority on projects of diversified size and conditions. In addition the JACKSON Line includes external vibrators for general construction, concrete pipe machines and vibratory table units for casting varied concrete products.

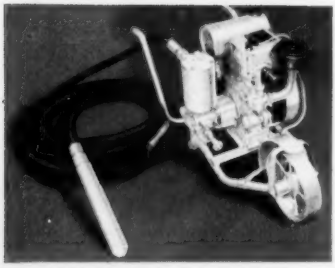
JACKSON HEAVY-DUTY VIBRATORY PAVING TUBE

For speedy full width airport and highway concrete paving. Up to 25 feet widths. Submergible dual tubes energized by powerful vibratory motors quickly transform harsh mixes to plastic state. Assures complete compaction — easy finishing. Cement savings up to 10% through reduction in W/C ratio. Attaches to any modern finisher. Variable frequency 3000-5600 V.P.M. Finger tip controls.



FS-6A GAS-DRIVEN, FLEXIBLE SHAFT CONCRETE VIBRATOR

For placing concrete in wall, column and slab sections of ordinary size. The smaller vibratory head is exceptionally well adapted to concrete placement in thinner sections. 3 H.P. air-cooled gas engine. Very fast. Automatic clutch. V-belt drive. Countershaft has oilite bearings. Turntable base, dirtproof. Optional wheelbarrow with drop down lifting handles. Flexible shaft drive. Vibrator head 2⅜" diameter x 18½" long or 1-13/16" x 17". Frequency 7000-72000 V.P.M. variable with length of shaft and concrete consistency.



HS-A1 HYDRAULIC CONCRETE VIBRATOR

This is a general purpose machine of the internal type adapted to a wide range of applications. Operated by light oil pumped through hose line 34 feet long to hydraulic motor in vibrator head. Valve on power plant adjusts frequency desired, from idling to top speed of 7200 V.P.M. All moving parts in the hydraulic medium (oil). Vibrator head 2¾" diameter, standard, gas engine, air cooled, 4.7 H.P. Selected by contractors for all types of concrete construction.

ELECTRIC TAMPER & EQUIPMENT CO.,

CONCRETE VIBRATORS

for all concrete applications

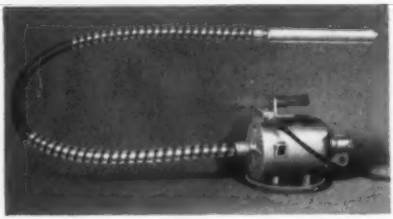
VS-4 VIBRO-SPADE *(Right)*

Designed for heavy duty placement of mass concrete. Ideal for large dams using mass concrete of very low slump characteristics or where coarse, large aggregate must be placed quickly. Submergible vibratory motor encased in chrome nickel steel. Operates on 110 and 220 volt, 3 phase circuits, (specify voltage desired), 1½ H.P. input rating. Speeds from 3600 to 4800 V.P.M. Overall length 5'. Length of vibratory unit 18½". Diameter of vibratory unit 6½". Straight rigid type handle complete with 2 circular lifting handles. Control switch mounted on top of handle. 25 to 40 cubic yards of mass concrete per hour.

VS-E2 VIBRO-SPADE

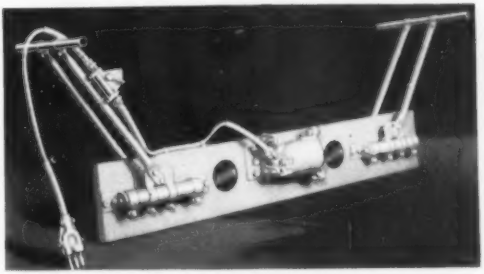
Suited to heavy general concrete construction such as foundations, thick walls, abutments and small dams. Bent rigid handle permits puddling concrete adjacent to side forms and transverse joints in pavement constructions. Submergible motor, vibrating head 4" in diameter, 18" long. Operates on 110 volt, 3 phase, 60 cycle power at 3600 V.P.M. Speed increased to 4000 V.P.M. when operated from JACKSON Power Plant. With frequency changers speed can be incrased to 4800 V.P.M. Straight or bent short rigid handles; also 12" or 26" flexible hose handles. 15 to 25 cubic

FS-5A ELECTRIC-DRIVEN, FLEXIBLE SHAFT CONCRETE VIBRATOR *(Left)*



Shaft, coupling and vibrator head same as FS-6A. Powered by 110 volt A.C.-D.C. Universal motor, rated ¾ H.P. at 7000 R. P. M., fan-cooled. Frequency 7000-7200 V. P. M. Bowl type base, slides easily. Regular shaft lengths —7, 14 or 21 feet.

SC-4A VIBRATORY HAND SCREED



For slab, deck, floor, ramp, walk or highway paving where use of hand screed is indicated. Any wood plank up to 20 feet can be used. Handles quickly attached and detached. Vibratory motor mounted to side of plank as illustrated. Requires 110 or 220 volts, 3 phase, 60 cycle A.C. (Specify voltage.)

LUDINGTON, MICHIGAN

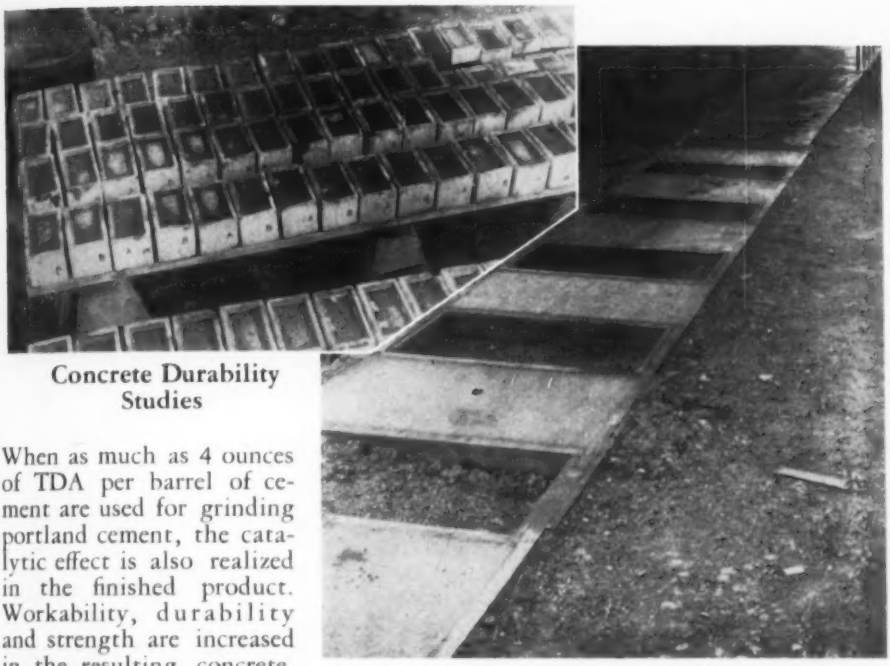
SUCCESSFUL APPLICATION OF COLLOID RESEARCH TO PRODUCTS OF THE CEMENT AND CONCRETE INDUSTRY

★ ★ ★

The Dewey and Almy Chemical Company was founded in 1919 on the premise that there was room for a supplier to industry who had a product that he was willing to back up with cooperative service and a continuous research program to improve and make the product more useful to the industry served.

Starting as suppliers of can-sealing compounds to the tin container industry, the resulting research program developed many by-products that find application in other fields such as shoe, textile, packaging, medical, general chemical, synthetic rubber, meteorological balloons, etc. For example, work to improve colloidal suspension led to the discovery of a new agent possessing unusual dispersing properties. At about the same time some work was being carried on as to the effect of carbon black in portland cement. Using the new dispersing agent with the carbon black gave unusual and unexpected results. The tinctorial value was markedly improved while the properties of the resulting concrete, including compressive strength and durability, were not impaired. That was our introduction to the portland cement field.

Continued research developed an improved catalyst dispersing agent that possessed outstanding properties. TDA, as it was named, proved to be a powerful grinding aid for the production of portland cement. When grinding cements of normal surface area, the use of between 1 and 2 ounces per barrel of cement resulted in a substantial increase in production and consequent power saving. When grinding cements of high fineness, the use of between 3 and 5 ounces per barrel of cement resulted in outstanding mill performance.



Concrete Durability Studies

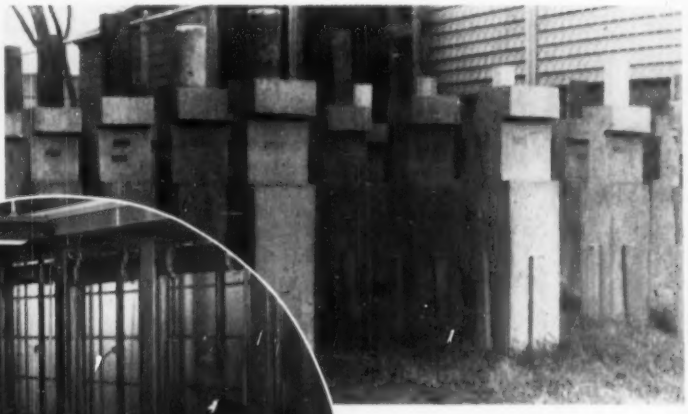
When as much as 4 ounces of TDA per barrel of cement are used for grinding portland cement, the catalytic effect is also realized in the finished product. Workability, durability and strength are increased in the resulting concrete. When larger amounts of TDA are incorporated in the cement, that is, up to 12 ounces per barrel, these desirable properties are still further enhanced. It is worthy to note that since its introduction to industry, TDA has been used in over a hundred million barrels of cement.

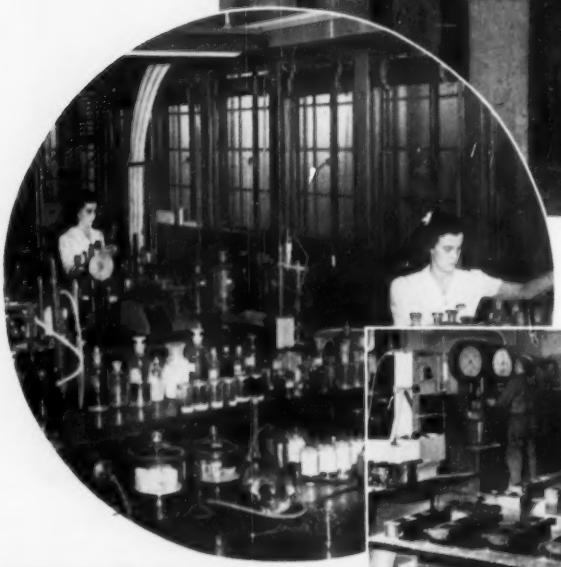
Experience gained from the use of TDA as a grinding aid in portland cement prompted the possibility that an efficient and economical grinding aid could be made for the grinding of cement raw materials (dry process). As a result, RDA was introduced to the cement industry and has been used extensively.

Early in our research with dispersing agents in concrete, the effect of entrained air in improving durability and plasticity was noted but the reduction in strength and density seemed to the industry at that time to offset these benefits. But in recent years, interest in the possibility of increasing concrete durability by means of air entrainment has been revived, largely because of the scaling effects of $CaCl_2$ used to de-ice pavement surfaces.

From our earlier work and drawing on our knowledge of the beneficial effects of TDA, a research program was undertaken to produce an air entraining agent which would yield concretes with fixed amounts of entrained air without appreciable loss in strength. The result was Darex AEA, a material which entrained controlled amounts of air, increased plasticity and workability and improved durability as measured by freezing and thawing, without loss in compressive strength. This material has been accepted by the American Society for Testing Materials in Specification C-175-44T. Darex AEA is interground in the cement clinker or is equally effective when added at the mixer and has been successfully used in a number of concrete structures.

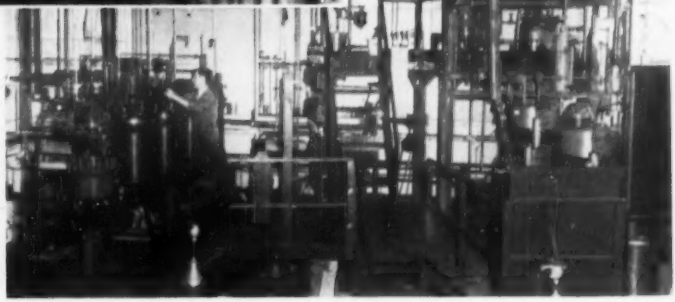
Studies of Alkali-Aggregate Reaction in Concrete





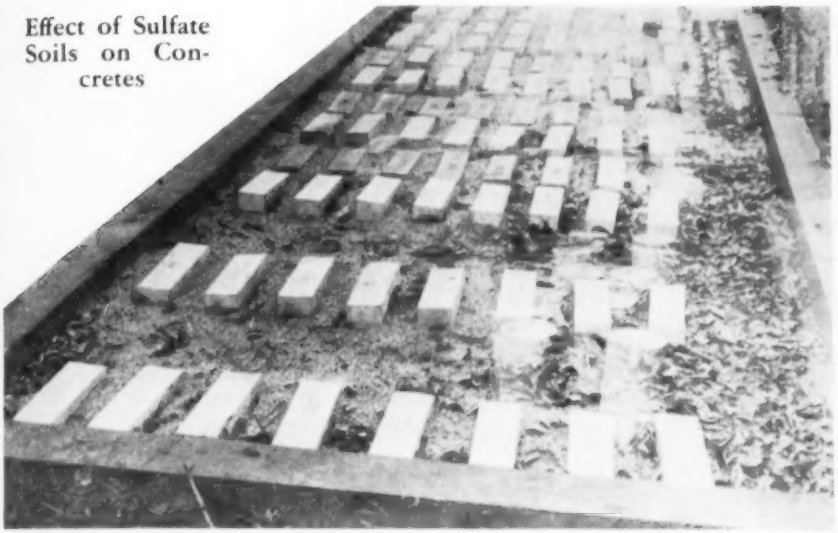
Typical Laboratories

Concrete Curing Room



Pilot Plant

Effect of Sulfate Soils on Concretes



In any research program on concrete, the question of controlled curing is important. It was therefore natural for us to use the research work of another division of the company which was working with the control of the transmission of moisture-vapor for packaging purposes, all the way from quick-frozen peas to dehydrated eggs. Their problem was to control the passage of moisture-vapor to keep it in or out, depending upon the product packaged. Curing of concrete involves the same principle. It was only necessary to adapt the form of the protective coating material to fit the need of curing concrete. The result was Daraseal, a highly efficient clear membrane curing compound which is finding wide use on concrete structures.

Each of these products represents the research approach to the problems of industry—finding a need, backing the need with intensive and continuous research and following up with cooperative service.

★ ★ ★

DEWEY AND ALMY CHEMICAL CO.

CAMBRIDGE 40, MASSACHUSETTS

TDA RDA

DAREX AEA

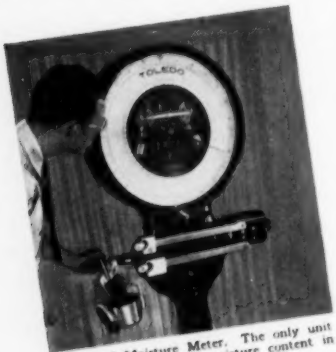
DARASEAL

Does this make sense to a Concrete Engineer?

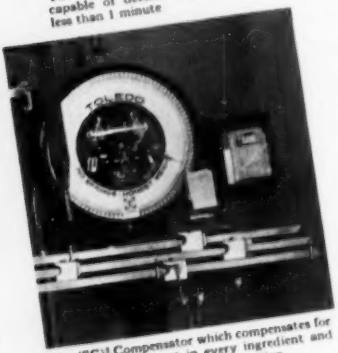
Is it worth while for an engineer to spend his time and effort in preparing a scientific concrete mix design, based on accurate laboratory tests, unless he can be sure of reasonably accurate batching and proportioning?

Concrete engineers have made mix design a science but good mix designs can be, and often are, ruined by inaccurate equipment and methods in the batching plant.

It is no longer necessary to put up with this condition. Specify [SC]² PRECISION CONCRETE CONTROL and batching accuracy is assured. It provides control that approximates laboratory accuracy and includes:



The (SC)² Moisture Meter. The only unit capable of determining moisture content in less than 1 minute



The (SC)² Compensator which compensates for the moisture content in every ingredient and graphically records its every operation

MOISTURE DETERMINATIONS, accurate to ¼% made in one minute; one for every batch when necessary.

AUTOMATIC COMPENSATION FOR MOISTURE CONTENT in fine and coarse aggregates. Does not require calculations, charts or tables. With it, any intelligent weigh-man can produce remarkable uniformity.

PRECISION PROPORTIONING OF ALL INGREDIENTS using TOLEDO, the world's most accurate scales. Automatic cut-off is used for cement and water and is optional for aggregates.

AUTOMATIC GRAPHIC RECORDS OF EVERY BATCH showing the delivered weight of each ingredient.

[SC]² CONTROL is approved by every concrete engineer, ready-mix operator and contractor who has ever used it.

So it's up to you. If you want your mix designs carried out accurately specify [SC]² CONTROL, by name or by performance.

Write us for details on how this can be done.

Scientific Concrete Service Corporation
McLachlen Building **Washington, D. C.**

SCIENTIFIC [SC]² CONCRETE

TRADE MARK REG. U. S. PATENT OFFICE

you can **protect** exterior

masonry surfaces

with non-critical materials . . .

The A. C. Horn Company announces *Waterfoil,* a scientific contribution to masonry protection Ten years of development are back of this product, including application on many structures under varying climatic conditions. *Waterfoil* is now available for general use.

No priorities are needed for *Waterfoil.* It is manufactured of non-critical materials . . . irreversible inorganic gels. *Waterfoil* is not a paint . . . it contains no linseed oil . . . no resin emulsion . . . casein or cement. It hardens into a heavy coating of microscopic "spongelike" character. Water vapor finds exit, but actual water penetration is impeded, thus helping to prevent reinforcing bar rust and concrete spalling.

Waterfoil becomes an integral "welded" part of the masonry surface to which it can be applied by any careful workman. No primers are used. If your masonry structures, brick, concrete or stucco, need decorative restoration and protection, get the details on *Waterfoil.* Backed by a nation wide company with 48 years of experience. Write today for literature.

Established 1897

A. C. HORN COMPANY

BUILDING MATERIALS DIVISION
Long Island City (1), New York

A HORN PRODUCT

WATERFOIL

THE UNIQUE TREATMINT FOR EXTERIOR MASONRY SURFACES

CUT COST *and* SPEED RUGGED REX

REX MIXERS SAVE TIME AND MONEY

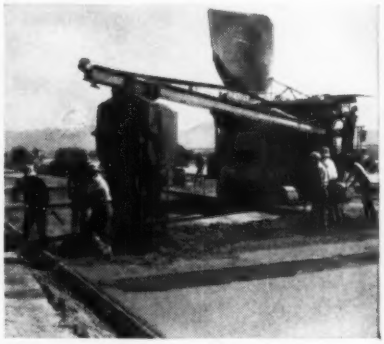
There's a fast, reliable Rex concrete mixer for every construction job—from basements to bridges—homes to factories. Designed and built for efficient, economical service, Rex Mixers assure higher quality concrete—more yards per day—at lower operating costs. Available in 3½S, 7S, 10S, 14S and 28S sizes.

REX MOTO-MIXERS DELIVER BETTER CONCRETE AT LOWER COST

Proved superior by actual performance from coast to coast, Rex Moto-Mixers deliver specification concrete thoroughly mixed and at lower cost. Sturdily built for greater endurance, Rex Moto-Mixers last longer . . . cost less to operate and maintain. They are available in all standard sizes.

REX PAVERS ARE BUILT FOR TOUGH JOBS AND TIGHT SCHEDULES

The dependable performance—the built-in stamina . . . the ruggedness . . . the smooth, fast operation of the Rex Paver—make it possible to complete jobs on time . . . at a profit. Operation is controlled by the Rex Mechanical Man, resulting in the fastest possible mixing cycle.



REX CONSTRUCTION

CHAIN BELT COMPANY

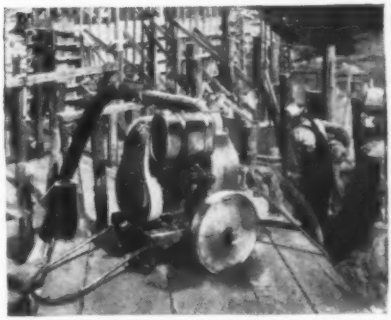
CONSTRUCTION *with* EQUIPMENT

REX PUMPCRETES TRANSPORT, ELEVATE & PLACE CONCRETE IN ONE OPERATION

Two valves, a piston, a cylinder and a re-mixing hopper are basically all there is to a Pumpcrete . . . the pump that *pumps* concrete by taking it from the mixer and *piping* it to forms wherever they are located. This means simplification and closer coordination of concreting operations on many difficult jobs.

REX SPEED PRIME PUMPS ARE FASTEST PRIMING

Rex self-priming, centrifugal pumps practically operate themselves. All you do is start the motor. No long waits for water to start moving . . . will continue to pump even with a leaky suction line. Rex Pumps are built for all types of general dewatering and supply on construction jobs. They are efficient and economical, with capacities ranging from 3,000 up to 125,000 gallons per hour. Gasoline or electric-power operated.



Write for Complete Information on REX CONSTRUCTION EQUIPMENT

Rex Engineers are specialists in the design, manufacture and application of equipment for mixing, hauling and placing of concrete and the moving of water. Write for illustrated booklets containing the complete facts and engineering data. Address Chain Belt Company, 1713 West Bruce Street, Milwaukee 4, Wisconsin.

MACHINERY

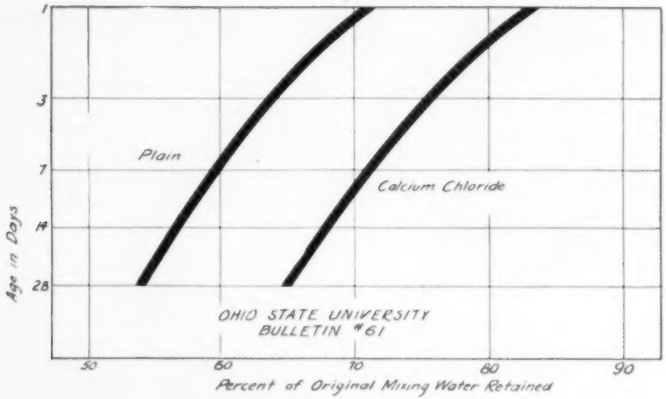
of MILWAUKEE

CALCIUM CHLORIDE
PROVIDES
"BUILT-IN" CURING

CALCIUM CHLORIDE, placed in with the aggregates in concrete, automatically and economically provides for the curing.

CALCIUM CHLORIDE in concrete

● INSURES MOISTURE FOR MAXIMUM HYDRATION OF THE CEMENT.



EFFECT OF CALCIUM CHLORIDE INTEGRAL CURING ON THE MOISTURE CONTROL OF CONCRETE

● ACCELERATES THE RATE OF HYDRATION OF THE CEMENT

● PROVIDES GREATER EARLY AND MAXIMUM STRENGTH OF CONCRETE.

The National Bureau of Standards reports*—"all concretes containing calcium chloride had greater strength at all ages tested," and that "Not only were the **early strengths greatly increased** by the addition of calcium chloride but the **one year strengths were appreciably increased.**

● INSURES MAXIMUM QUALITY CONCRETE.

By speeding up and maximizing the hydration of the cement Calcium Chloride in the mix produces **greater strength concrete.** Every particle of cement responds to this integral curing reaction.

This very simple and economical method of curing obviates the necessity of detailed inspections. And much equipment, material, and labor are saved due to the gain in Early Strength of the Concrete.

Calcium Chloride gives these benefits with all cements—standard portland, high early strength and white cement. Calcium Chloride is of particular value when used with air-entraining cements as it compensates for their tendency to lower early strengths.

* See "Effect of Calcium Chloride on Portland Cements and Concretes" by Paul Rapp; Proceedings Highway Research Board, Dec. 1934.

Write for specifications. Our engineers will be glad to answer any questions addressed to the

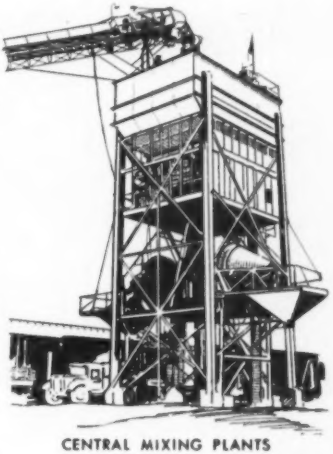
CALCIUM CHLORIDE ASSOCIATION

4145 Penobscot Bldg. DETROIT 26, MICH.

After The WAR

THE MOST IMPORTANT MATERIAL WILL BE

★ CONCRETE ★



CENTRAL MIXING PLANTS

That is not to say that concrete has not been of paramount importance *during* the war — the point, however, and a vital economic point, — is that in the years after the war there will be a new and accelerated tempo in concrete construction. The pace will demand and produce new ideas and new methods of handling to contribute to a greater efficiency.

Butler builds equipment *today* that anticipates the standard designs of tomorrow. Let "Butler Engineered Design" help you lead in building the future — in concrete.

CONCRETE PRODUCTS PLANTS

Engineered BUTLER *Design*

CARSCOOP
BIN GATES
BATCHING PLANTS
READY MIXED CONCRETE PLANTS
CENTRAL MIXING PLANTS
BULK CEMENT PLANTS
CONCRETE PRODUCTS PLANTS
WEIGHING BATCHERS
PORTABLE CONCRETE HOPPERS
CEMENT TRANSPORTS

Butler Bin Company

WAUKESHA, WISCONSIN

THE KELLEY METHOD DELIVERS A SURFACE—CREATES A SAVING IN

The Kelley Compactor Power Float at work on dry topping thirty minutes after placing. The weight of the machine, plus the action of the compactor hammers and the rotating of the disc, compacts and floats out the topping to a smooth, level surface, bringing just enough mortar to the surface for the troweling operation.

LEADING AUTHORITIES RECOMMEND AND SAY THAT:

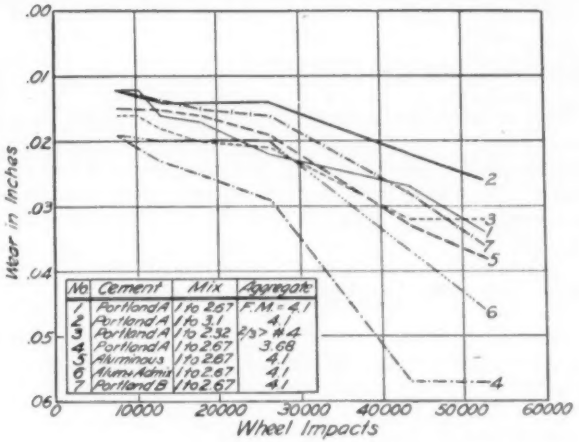
Low water-cement ratio mixes using graded aggregate give high strength, water tightness —resistance to wear—prevent segregation and allow leaner mixes.

Lean mixes of low water content have less shrinkage than rich mixes of high water content.

A concrete wearing surface using this mix has the coarse aggregate in the surface to take the wear; eliminates dusting caused by excess water bringing fines to surface and eliminates cracking and crazing caused by water shrinkage and excess fines.

KELLEY ELECTRIC MACHINE COMPANY

SUPERIOR WEARING CONCRETE LABOR AND CEMENT COSTS



Wear of Seven Sample Segments of Test Floor

Reference: Journal of American Concrete Institute, Nov.-Dec. 1934 & Sept. 1938

MIX No. 2—**Leanest in cement content—best wearing qualities.**

MIX No. 4—**Richer in cement content and 12% more fines passing 50 mesh sieve—poorest wearing qualities.**

The coarse aggregate at surface takes the wear.

Excess fines and cement lower wearing qualities.

The Kelley Compactor Power Float provides a mechanical means for working the recommended, harsh, dry mixes and delivers a dense, durable, level, dustless hard-wearing surface at a saving in labor and cement costs. The rotating disc moves the material and delivers an even, floated surface. At the same time, the weight of the machine, plus the 1200 per minute hammer blows break down the bridging of the aggregate, drive out impacted air and densify the mix throughout its thickness without segregation, leaving no room for shrinkage—the cause of cracking.

The Kelley Compactor Power Float is built in both Gasoline and Electric Models for all types of concrete wearing surfaces:

FLOORS - - - - - DRIVEWAYS - - - - - PAVEMENTS - - - - - ROADS

Booklet, "Concrete Facts about Concrete Floor Finishes" and full information sent on request.

BUFFALO, (Zone 17) 287 Hinman Ave. NEW YORK

A few of the ACI special publications in large current demand as contributions to technical progress

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members. $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information about ACI Membership and Publications address

AMERICAN CONCRETE INSTITUTE

742 New Center Building DETROIT 2, MICHIGAN

HUNT PROCESS

Liquid Membrane cures have become universally known and have proven so satisfactory, that now they are generally accepted as a proper method for curing Portland Cement Concrete.

About the only problem remaining is to determine whether a curing compound has the proper qualities to accomplish the desired curing result. The principal quality to be determined is whether the film resulting from the compound application to the concrete is sufficiently impervious to insure the retention in the concrete of sufficient mixing water to positively and completely hydrate the cement. Other qualities are essential such as adhesion, color, drying properties, sprayability and the possible deleterious effect to the concrete itself, but these can be rather easily determined by observation or some simple testing procedure.

There is no simple field method of determining the sealing qualities. The solid content of the compound, as such, means little, for it is the film forming characteristics of the solid ingredients which perform this function. Compounds with as high as 60 percent solid content have been found to have little, if any, sealing qualities.

Many engineers have recognized the necessity of determining the sealing qualities of a curing compound and in the last few years, a number of testing methods have been advanced. Most of these methods are satisfactory, but some necessitate the use of rather expensive equipment. However, most laboratories are equipped or could be equipped without too much expense to perform satisfactory tests.

Our Research Laboratory finds that the ASTM method designation C-156-40T contains a most efficient method for determining the sealing quality. However we believe some modifications are necessary when used merely for the purpose of determining a curing compound's sealing quality. We believe the proportioning of 1 part cement, 2.2 parts Ottawa sand, and 0.4 parts water will give a mix sufficiently close to that obtained by the "flow table" so that this step in the procedure can be eliminated. As the desire is to determine the sealing quality of the curing compound, the percentage of water loss should be based on the water in the specimen at the time the compound is applied and not at the time the specimen is made. The loss in weight of the specimen between time of mixing and time of application of the compound is loss of evaporated water, and the deduction of this weight from the original weight of water in the specimen gives the water remaining in the specimen at the time of application.

Our laboratory also finds that when using the 2" thickness specimen, molded in a glass mold and applying the curing compound at a rate of 200 square feet per gallon, that a compound showing less than $3\frac{1}{2}$ percent water loss in a 3-day test period is a satisfactory cure.

Due to the nature of conditions brought on by the war, it has been necessary to pour concrete under all, and varying climates, from intense cold to extreme heat. Hunt Process cures have been used in all of these cases and proven most satisfactory.

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HUNT PROCESS COMPANY

The Original Membrane Concrete Curing Compound

Main Office and Plant, 7012 Stanford Avenue, Los Angeles 1, California

Hunt Process Clear—Hunt Process Black

THIN SHELL CONCRETE ROOFS

SPEEDY, ECONOMICAL, ADAPTABLE, TIME TESTED, FIRE PROOF

LOW MAINTENANCE AND INSURANCE RATES

Concrete forms roll on own wheels for greater reuse and reduction in cost.
Smooth underside of roof and unobstructed useful space.
Fewer columns and greater spans.
Beam action of curved roof utilized.
Local materials and local labor used.

Industrial Plants, Garages, Warehouses, Auditoriums

ROBERTS AND SCHAEFER COMPANY
ENGINEERS

307 North Michigan Avenue **CHICAGO 1, ILLINOIS**

Mall

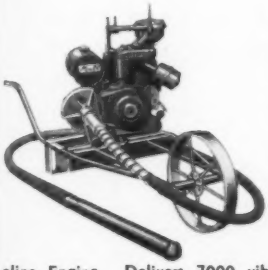
REG. U.S. PAT. OFF.

Concrete Vibrators

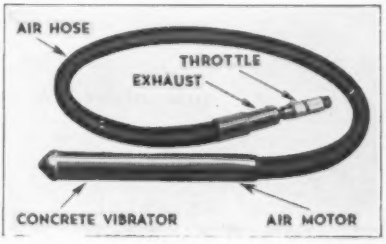
A SIZE AND MODEL FOR EVERY JOB

MALL Concrete Vibrators offer many economical and structural advantages unattainable by hand placing methods. They permit the use of stiffer concrete mixtures. They place concrete faster and better with less labor. They permit placing in areas inaccessible for hand methods. They make possible the accurate scheduling of operations. In addition, they eliminate honeycombs, voids and expensive hand patching, and assure a better bond with reinforcement, a better bond between successive layers, lower absorption, greater density and watertightness and increased compressive as well as flexural strength. For their size MALL Vibrators place more concrete than any other vibrator.

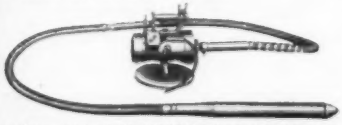
MALL CONCRETE VIBRATOR powered by 1½ H. P. Variable Speed, Air Cooled, Gasoline Engine. An easily portable, all-purpose tool. Delivers a continuous flow of power for 7 quickly interchangeable attachments for CONCRETE VIBRATING and SURFACING, FORM SANDING, WIRE BRUSHING, SHARPENING TOOLS, DRILLING in concrete, brick, iron and steel. Operates all day on very little fuel. Easy to keep busy.



MALL CONCRETE VIBRATOR powered with 3 H. P. Gasoline Engine. Delivers 7000 vibration frequencies per minute. Has variable speed, single cylinder, four cycle, air cooled gasoline engine that runs all day on 1½ to 2 gallons of gasoline. The wheelbarrow type mounting provides unusual portability. Interchangeable attachments can be furnished for CONCRETE SURFACING, WIRE BRUSHING, SHARPENING TOOLS and BITS, FORM SANDING, DRILLING in concrete, brick, iron or steel.



7500 r.p.m. PNEUMATIC MODEL. For placing concrete in tunnels, caissons, and other deep construction. All parts are easily renewable in field and no special tools are required. Equipped with twist-hand throttle for easy operation.



MALL 1½ H. P. UNIVERSAL ELECTRIC CONCRETE VIBRATOR. DELIVERS 9000 vibration frequencies per minute under load. Available with 14 ft., 21 ft., or 28 ft., of flexible shafting. Has low round base mounting and convenient carrying handle.

Immediate Delivery on Suitable Priority.
Write for Literature and Prices.

MALL TOOL COMPANY

7703 South Chicago Avenue, Chicago, 19, Illinois
Offices in Principal Cities

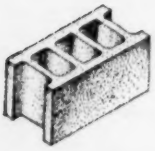
ARMY E NAVY

Mall REG. U.S. PAT. OFF. PORTABLE POWER TOOLS

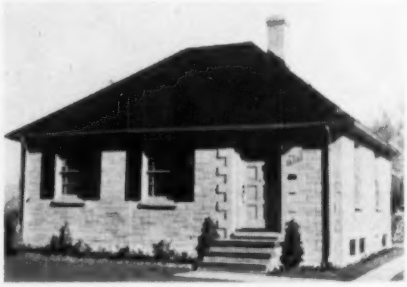
BESSER Vibrapac BETTER Concrete Masonry Units
THE MODERN BUILDING MATERIAL

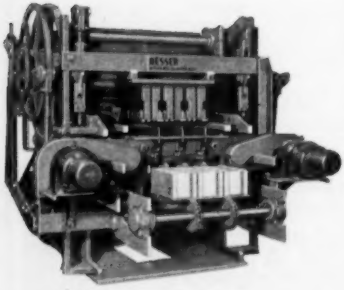
OLD In Proved Durability
NEW In Design And Effect

BESSER Vibrapac BETTER Concrete Masonry Units have established a new height of perfection in Concrete Masonry. Vibrapac units can be controlled in density and texture for any special modern requirement in exterior and interior use. Adapted to all types of buildings and all styles of architecture.



BESSER Vibrapac BETTER Concrete Masonry Units in the hands of skilled architects and builders have become in effect a new building material.





Besser Super Automatic Plain Pallet Vibrapac. Capacity 600 - 8" x 8" x 16" per hour made 3 at a time on one Plain Pallet. Smaller units made in larger multiples on the same pallets.

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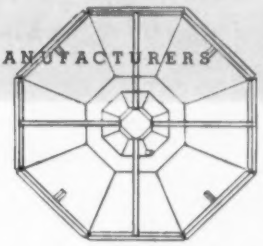

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Our experience record since 1922:

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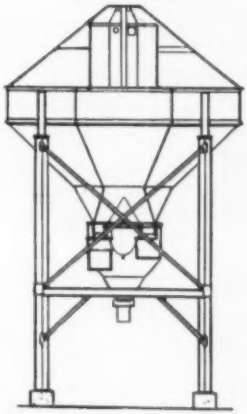

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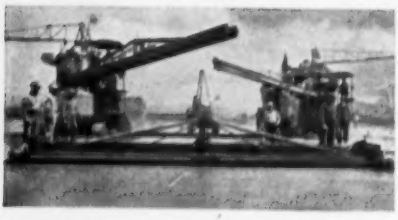
THE C. S. JOHNSON COMPANY, Champaign, Illinois

MASTER VIBRATORY FINISHING SCREED

- **How it provides accurate strike-off and compaction in a single easy operation.**
- **How it finishes over 6,000 sq. ft. per hour.**



Note. Hand-drawn Vibratory Finishing Screed. In the main photograph, control throttle at right end provides easy regulation of vibrating speed. The small illustration above shows a 26' Vibratory Finishing Screed working in California on a 25' pavement strip of 11" thickness . . . and finishing strip in one pass. Pavers are 34-E single and 34-E dual drum.

Master Vibratory Finishing Screed works only part time to take care of pavers' output. Moves forward at 6' per min. One screed should handle two dual drum pavers for maximum efficiency on this type of strip. Reports indicate that finishing done by Master screed equals two screed type tamper-equipped finishing machines obtaining better compaction and requires less cement finishers.

★ ★ ★

Master 25' Vibratory Finishing Screed, below on the job at San Bernardino, Cal. Note, Screed is attached to box spreader by two cables.

PURPOSE AND RESULTS OBTAINED THEREBY

Here, at last, is a machine so simple that the average laborer with little mechanical knowledge can understand . . . yet get desirable results never obtained before by other methods. The Master Vibratory Finishing Screed is so designed that through the application of vibration on concrete slabs, maximum density, strength and minimum surface variations are obtained. It also allows the placing of concrete with low water content, thereby gaining greater internal strength in the concrete structure.

The Screed in striking off operation rests on pre-set forms or guides and leaves the surface of the slab with corresponding true alignment The Master Screed vibrates uniformly throughout its entire length up to maximum sizes built, assuring uniform compaction and eliminating the element of error due to labor.

The light weight of the Master Vibratory Finishing Screed makes it satisfactory for manual operation on such work as: Highways, concrete floors and other types of slabs; Airport runways, aprons, hangar floors; Warehouse floors; Dock decks; City street paving; Sidewalks; Industrial plant drives; **Canal inverts; Bridge decks; Concrete roofs;** and many other applications. In addition, the Master Screed is the only highway slab finishing screed that can produce a true parabolic curve crown. All sizes of Master Screeds can be built for any type of crown or invert.

DESIGN: The Master Vibratory Finishing Screed is a simple, sturdy all-steel, vibrating strike-off screed, consisting of a steel Tee-shape and vibrating member spanning the slab, supported at either end by adjustable spring-mounted steel shoes which travel on the forms or guides between which the concrete is deposited.

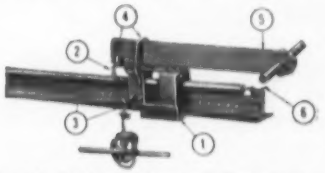
MASTER VIBRATOR FINISHING SCREED (Continued)

Centrally located is a vibrating element securely fastened to the Tee section and driven by an **air cooled gasoline engine** or electric motor through a Master automatic clutch and Vee belt drive. Vibrations are adjustable from 2000 to 6400 per minute through the use of the engine throttle, thereby providing means of applying the number of vibrations per minute that produces the most satisfactory results in the concrete being used. The spring-mounted end shoes are adjustable for various widths by close drilled holes in the main member (see diagram at right).

The screed is furnished with Draw Ropes at either end for forward or backward movement. Generally on units up to 16 feet long, two laborers are all that are required. Longer screed may require up to four laborers. Weights have been reduced to the minimum, a 20 foot Vibratory Finishing Screed weighing 380 pounds and smaller sizes correspondingly lighter. Movement of the screed from one location to another is quickly made by use of the carrying handles furnished with screed.

Vibratory screeds to finish slabs of 6 ft., 10 ft., 13 ft., 16 ft., 20 ft., 25 ft., and 26 ft. are standard—longer lengths built to order. All standard screeds adjust down 7' 6" from maximum length, except VS-6 (6' screed), which adjusts down 3' 6". More adjustment can be obtained by drilling vibratory beam in field.

For further details write for Master Bulletin 596



1. Adjustable spring mounted shoes. 2. 2-way draw. 3. Easy adjustment for length. 4. Yoke for lifting. 5. Lifting handle. 6. Remote engine control.

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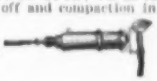
☐ Gas or Electric Concrete Vibrators to meet every placing requirement

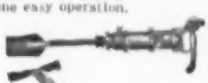
☐ Concrete Surfacing Attachments for all needs

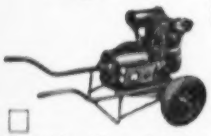
☐ Full line of Flood and Shovel Lights (not illustrated).

☐ Vibratory Concrete Finishing Screeds—Models: VS-6 ft.; VS-10 ft.; VS-13 ft.; VS-16 ft.; VS-20 ft.; VS-25 ft. and 26-ft., adjustable for lengths—strike off and compaction in one easy operation.



☐ Electric "Power Blow" Hammer or Spade and Tools for heavy or light duty work.





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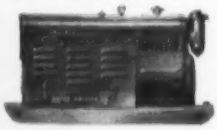

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SYNOPSES of RECENT ACI PAPERS and REPORTS are available for the asking - - - - below is a small sampling only from JOURNAL issues of the current volume year - - - - - - -

[available in separate prints at 25¢ cents each except as otherwise noted; starred titles are 50 cents each.]

THE EFFECTIVENESS OF VARIOUS TREATMENTS AND COATINGS FOR CONCRETE IN REDUCING THE PENETRATION OF KEROSENE..... 41-2

F. B. HORNIBROOK—Sept. 1944, p. 13-20 (V. 41)

Measurements were made of the penetration of kerosene under a 12 foot pressure head into discs of concrete which had received various treatments or coatings. The tests were classified into 7 groups as follows: (1) reference concrete, (2) integral admixtures, (3) sodium silicate and magnesium fluosilicate treatments, (4) plaster coats, plain and with admixtures, (5) magnesium oxychloride type coatings, (6) linseed oil and spar varnish coatings, and (7) synthetic plastic and latex coatings. Comparisons of the relative rates of penetration of kerosene into the specimens of each group are given, together with a discussion of various other properties of each group.

ADMIXTURES FOR CONCRETE..... 41-5

A REPORT BY ACI COMMITTEE 212—Nov. 1944, pp.73-88 (V. 41)

With the aim of providing a perspective of the field of admixtures for the use of the engineer confronted with a need of modifying concrete to meet special requirements of a given job, Committee 212 has classified admixtures into 9 broad groups. Discussions are given of the factors which might indicate the usefulness of admixtures of each group, and of the important effects which may ordinarily be expected from the use of materials of each group. The 9 groups are as follows: (1) accelerators, (2) air-entraining agents, (3) gas-forming agents, (4) natural cementing materials, (5) pozzolanic materials, (6) retarders, (7) water-repelling agents, (8) workability agents, and (9) miscellaneous.

THE EFFECT OF CURING CONDITIONS ON COMPRESSIVE, TENSILE AND FLEXURAL STRENGTH OF CONCRETE CONTAINING HAYDITE AGGREGATE..... 41-7

E. B. HANSON, JR. and W. T. NEELANDS—Nov. 1944, pp. 105-116 (V. 41)

Lightweight concrete has been given a severe test in the U. S. Maritime Commission's present concrete ship construction program. In its use, problems arose that could not be solved by the application of sand-gravel concrete data. This paper describes some of the strength characteristics of this type of concrete. Data herein are consistent in showing that rapid moisture loss from Haydite concrete produces a serious retrogression in the tensile and flexural strengths, regardless of the length of moist curing. This decline in strength, caused by drying shrinkage stresses developing in the outer fibers as the moisture content becomes unbalanced, is of a temporary nature and apparently can be curbed by the application of paint or membrane seal following the moist curing period. The drying shrinkage may well contribute to serious cracking in some types of structures if control is not maintained.

CONCRETE OPERATIONS IN THE CONCRETE SHIP PROGRAM★ 41-9

LEWIS H. TUTHILL—Jan. 1945, pp. 137-180 (V. 41)

This paper describes only briefly the hulls constructed in the concrete ship program of the U. S. Maritime Commission but goes into more detail in connection with problems encountered and their solution in the course of these concrete operations. Construction joint procedure, lightweight aggregate concrete and mix control, handling, placing and vibration practice, curing, testing and repair problems are described in the belief that much of this information is applicable to any concrete work of high standard. Design of hulls is not discussed except as construction is affected. The ships have not been in service sufficiently long to justify much discussion of their performance or durability.

AN INSTRUMENT AND A TECHNIC FOR FIELD DETERMINATION OF THE MODULUS OF ELASTICITY, AND FLEXURAL STRENGTH, OF CONCRETE (PAVEMENTS).....41-11

BARTLETT G. LONG, HENRY J. KURTZ, and THOMAS A. SANDENAW—Jan. 1945, pp. 217-232 (V. 41)

An instrument for determination of the dynamic modulus of elasticity of concrete, in situ, is described. Test results are presented which show (a) the comparison of test values of E, obtained by various older methods, with that obtained with the new instrument; and (b) the relationship of such values to the flexural strength of concrete. It is concluded that adoption of the new method and technic is justified; and that widespread use of the new instrument would eliminate the necessity for casting field specimens during construction (except perhaps for day-to-day control purposes) or of removing costly "samples" from completed works. A method for determining the thickness of concrete pavements is briefly discussed. A rather extensive bibliography is included.

[—while the complete papers and reports are for sale, the synopses pamphlets, providing a survey of recent ACI work, are cheerfully sent on request - - - address]

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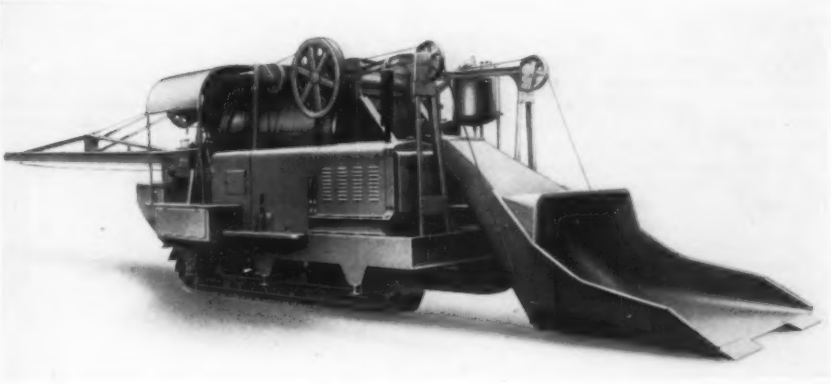
STEADY, ACCURATE WATER SUPPLY to a paver mixing drum is essential if a uniform concrete consistency is to be maintained throughout a job.



RANSOME PAVERS are equipped with a gravity feed water system that is absolutely accurate and fool proof. It has a non-pressure, syphon type measuring device that measures the batch water accurately, without variation or stoppage of the water flow. Ransome engineers have made this possible by producing a tank which is carefully calibrated, not effected by grades, side inclines, or variations in line pressure. This is one feature of Ransome Pavers. There are many others.

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PLASTIMENT

ACTION and BENEFITS

ACTION

PLASTIMENT is a dry powder, added to concrete at the mixer in a proportion not exceeding 1% by weight of the cement. It contains agents which cause certain physical and chemical reactions:

These agents slightly increase the surface tension of the gaging water thus destroying emulsions and foam occurring during mixing.

They disperse the cement particles throughout the mass, thus insuring greater utilization of a given quantity of cement.

They delay formation of gels, ordinarily formed immediately upon contact of the highly reactive surface of the cement grains with water, and form mono-molecular layers between the cement grains and the water.

BENEFITS

Water cement ratio:

Gels are usually formed immediately and require a certain amount of water for their formation. In PLASTIMENT concrete, this water remains free for lubrication, and allows a reduction of the water cement ratio with all its advantages without impairing the workability. The degree of the reduction depends upon the type of cement and the aggregates used.

For the same water cement ratio the workability of the concrete is greatly improved and at the same time the density is increased. A higher quality concrete is obtained in addition to the better placeability.

Hardening process:

Cement grains, coated with the mono-molecular layer of PLASTIMENT, hydrate at an initially delayed rate. The setting time, therefore, is extended, and the strength of the concrete lower during the first 24 hours. Then, when the reaction has started throughout the grains, the rate of hardening proceeds normally. Due to the tighter packing, higher strengths are eventually obtained.

Strength at 24 hours is about 20% lower than for plain concrete. At 48 hours concrete with and without PLASTIMENT have about the same strength. At 7 days PLASTIMENT concrete is about 20% stronger and over 30% at 28 days and longer periods.

The heat developed by hydration is highest (per unit of time) just before and soon after the period of final set. By delaying the rate of hardening, less heat is developed per unit of time, the increase in temperature is less, due to radiation over a longer period of time and, therefore, less internal stress occurs in the concrete.

The total amount of heat developed by the hydration of a given quantity of cement is not reduced, but for the same total amount of heat a higher quality concrete is obtained.

Bleeding:

Detrimental only insofar as it is the cause of waterpockets, scum and laitance. Otherwise bleeding is beneficial by eliminating part of the surplus water needed for a certain workability and allowing tighter packing through settlement.

Shrinkage:

Numerous tests show that PLASTIMENT not only does not increase the shrinkage of cement or concrete but often actually decreases it. This favorable effect on shrinkage for a given quantity of cement, and the considerable increase in strength and the decreased heat of hydration per unit of time, are responsible for the reduction of shrinkage cracks in PLASTIMENT concrete.

Volume change results from alternate wetting and drying, and from temperature changes. Wetting and drying changes are dependent upon the degree of absorption. Watertight, dense, PLASTIMENT concrete will absorb less water, and, therefore, shows an impressive reduction in drying shrinkage.

Bond in construction joints:

Weather damage to structures usually originates in faulty construction joints. Weak construction joints are caused by scum and

laitance deposited on the surface of concrete by rising emulsions. PLASTIMENT concrete prevents the carrying away of the finest cement particles through bleeding and minimizes segregation. PLASTIMENT concrete bleeds clear water and, therefore, produces a surface to which subsequent pourings of concrete adhere well. When bending plain concrete specimens, they often break through the untreated joint itself, but PLASTIMENT concrete specimens usually break in the new concrete.

Cold joints:

PLASTIMENT will delay the setting time of concrete sufficiently to permit its placing (without cold joints) under difficult conditions; such as when pouring heavily reinforced sections, or when using pump concrete or rich mixes at higher temperatures.

Bond to reinforcings:

Reduction of initially formed gels allows the cement grains to lie closer to steel reinforcing. The dispersing action of PLASTIMENT causes the concrete to be more uniformly distributed around the steel. The bonding strength, therefore, is greatly improved. Official tests show increases of 20-80% of the bond to steel for vertical as well as horizontal reinforcing.

Strength:

Although high strength may not be of importance in most concrete work, it is generally considered to be an indication as to the other desired qualities of concrete. Tests show that PLASTIMENT increases the strength considerably even if the quantity of water is kept the same to obtain better workability. This indicates the advisability of its use in heavily reinforced sections, so that easy placement can be obtained without sacrifice in quality.

Surface hardness:

Due to the increased density and tighter packing and absence of scum and laitance, the resistance against abrasion is considerably increased, thus affording better protection against erosion. Increases range from 20-30%. This indicates the advisability for use of PLASTIMENT in heavy duty floors, roads, spillways and tunnel linings.

Resistance to frost:

The lower the water cement ratio, the less absorbent a concrete tends to be and, therefore, its resistance to climatic influences is higher (see under impermeability). Tests and actual experience in the field over a period of more than 10 years show that PLASTIMENT improves this quality of concrete considerably.

Impermeability:

A structurally sound wall made of properly graded, designed, and placed concrete will withstand the usual water pressure encountered in the field. Leakages occur only through honeycombs, porous concrete, and through improperly bonded construction joints. By reducing segregation, improving workability, bond, and density, PLASTIMENT helps to produce a watertight structure.

CONCLUSION

PLASTIMENT is not a cure-all for concrete. It cannot correct nor compensate for improper design, gradation, placing or curing.

PLASTIMENT **WILL** enhance the vital qualities of a given type of concrete by improving its denseness, strength and workability; its uniformity and bond; its hardness and resistance to climatic influences and injurious solutions; and by decreasing its permeability and shrinkage. The use of PLASTIMENT is indicated and can be justified where any one of these qualities is required.

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Viber Company 381

ACI NEWS LETTER

Vol. 16 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1945

THE "MUCH CURTAILED" 1945 CONVENTION

As announced on *News Letter* page 1 in January, the ACI Board of Direction (prior to the recent more stringent regulations in discouragement of "conventions as usual") complied with an ODT request for cooperation in the reduction of civilian travel not necessary to the war effort. The 1945 convention was reduced to what seemed to be the barest of administrative needs—no technical sessions—for an estimated travel saving of 90 percent—fewer it is believed than 30 persons. Meetings at the Hotel New Yorker, New York City, February 14-16, are these:

Board of Direction: 12 o'clock, noon (at lunch) Wednesday, February 14, continuing through dinner hour into the evening.

Advisory Committee, Raymond E. Davis, Chairman; 10 a.m. Thursday, February 15, continuing through the afternoon.

Publications Committee, Douglas E. Parsons, Chairman: 6:00 p.m. (at dinner) Thursday, February 15, through the evening and continuing at 9 a.m. Friday, February 16 until 11:45 a.m.

General Session of ACI (the 1945 Convention): 12, noon (at luncheon), Friday February 16. Following the coffee: the report of tellers and induction of new officers and directors; an address by the retiring president; the award of Wason Medals (see January *News Letter*) and matters pertinent to the administration of Institute affairs. To insure the necessary quorum at the general session, dependence will be on New York City *Members only* plus the members of the Board and of the Advisory and the Publications Committees.

ACI IN WAR TIMES*

By the ACI SECRETARY-TREASURER

ACI accepts the uncertainties of these days as do thousands of other organizations devoted to some form of public service.

The American Concrete Institute is gratified that many thousands of its publications were available when needed for use by government agencies on construction projects necessary to the nation's defensive and offensive program.

In these days, when fighting a war comes first, one may still renew his energies in a glimpse of days to come for which the war is fought. We may look forward to a more nearly adequate staff (here in the home office) to keep our work up to date and to realize more of the abundant ACI potentialities —potentialities frustrated in turn by depression and by world conflict. We may look forward to more manpower to support a good will at our printers to get our publications out faster; to more abundant printing paper and a return to the pre-depression schedule of ten JOURNALS a year; to bindery conditions that do not delay our annual volume for months on end; to the growth of an all-important nucleous of "working" Members and they less hard-pressed by unusual calls upon their time and energies; to conventions, not curtailed by pressures of war necessity, that excel in technical interest and accomplishment and in the enjoyments of the comradeship of congenial minds. In fact, like many millions of others, we look forward to peace and to new and further horizons.

Meanwhile, it is the special gratification of these dislocated times to see, every day, how some are "stretching" themselves in effort—members of the Institute's small secretarial staff, long-tried or new to their tasks, who have a keener sense of their job's responsibilities than of its hours; and those stalwarts of our Membership who, with days already more than full, have still, in countless ways, done amazing service without which ACI effort must have floundered.

We wish them all an abundance of those mysterious, unanalyzable satisfactions that are distilled from living by the priceless urge that persists above and beyond any clock-timed, dollar-paid employment. It is such service, by a gallant few, which makes possible an organization of so-called "mutual effort." Such service sustains ACI through war, for the higher human values that war is to reclaim.

*Not submitted for Publications Committee approval.

Honor Roll

Feb. 1, 1944 to Jan. 29, 1945

As these pages go to press, two days before the end of the Honor Roll year, Lewis H. Tuthill has forged ahead to first place with 17 new Members sponsored, with F. E. Richart in second place with 13—but there are still two days to go.

Lewis H. Tuthill **17**
F. E. Richart **13**
Mehmet Tokay **8**
J. W. Kelly **6**
A. J. Boase **5½**
Jacob J. Creskoff **5**
William A. Jones **4½**
Harry Erps **4**
Rene Paulido Morales **4**
Douglas E. Parsons **3½**
Charles E. Wuerpel **3½**
J. A. Crofts **3**
Kenneth K. Knight **3**
F. W. Panhorst **3**
H. F. Gonnerman **2½**
Hugh Barnes **2**
Elmer B. Belt **2**
R. F. Blanks **2**
C. E. Everhart **2**
P. J. Freeman **2**
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Fred E. Hale **2**
W. G. McFarland **2**
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D. F. Roberts **2**
R. T. Sherrod **2**
Kenneth H. Talbot **2**
A. E. Cashe 1½
Raymond E. Davis 1½
H. B. Emerson 1½
Bengt. Friberg 1½
Albert Haertlein 1½
M. E. James 1½
Warren Raeder 1½
K. E. Whitman 1½
R. R. Zipprodt 1½
Owen Arthur Aisher 1
R. Howard Annin 1
L. J. Baistow 1
J. F. Barton 1
S. D. Burks 1
Morgan R. Butler 1
Julian B. Carson 1
S. J. Chamberlin 1
H. Clare 1
H. F. Clemmer 1
Joseph Di Stasio 1
B. M. Dornblatt 1
John J. Earley 1
Frederic Faris 1
H. D. Farmer 1
H. E. French 1
H. M. Hadley 1
Elliot A. Haller 1
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Fred Hubbard 1
Norman J. Huber 1
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Henry L. Kennedy 1
W. E. Lumb 1
Donald R. MacPherson 1
H. E. A. McCarty 1
Roscoe J. Mason 1
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Walter H. Price 1
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John R. Ruhling 1
M. K. Scheirer 1
Ralph A. Sherman 1
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Charles Snyder 1
Gregory P. Tschebotarioff 1
E. Viens 1
J. J. Waddell 1
Carl B. Warren 1
Geo. W. Whitesides 1
Benjamin Wilk 1
Chas. P. Williams 1
R. J. Willson 1
M. O. Withey 1
E. B. Wood 1
T. Van Dyke Woodford 1

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J. M. Breen
Geo. C. Britton
A. D. Ciresi

Herbert K. Cook
Theodore Crane
R. W. Crum
R. A. Crysler
G. E. Davis
R. E. Davis
Harmer Davis
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A. W. Dudley
John R. Dwyer
H. H. Edwards
W. J. Emmons
H. J. Gilkey
Homer M. Hadley
P. J. Halloran
Walter N. Handy
P. W. Helsley
M. Hirschthal
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N. M. Loney
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Warren Raeder
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Roy T. Sessums
R. H. H. Stanger
A. L. Strong
Sanford E. Thompson
Calvin T. Watts
Kurt F. Wendt
E. P. H. Willett
R. W. Winters
J. C. Witt

WHO'S WHO

T. C. Powers

an ACI Member since 1927, twice a Wason Research Medalist, is not by any means new to ACI *Proceedings* pages (see 1944 ACI Directory, p. 83) nor to this "Who's Who" department (see N. L. p. 2, June, 1939, coincident with the publication of his second paper—on "bleeding"—to win the Research Medal). A member of the Portland Cement Association research staff since 1930, he has been in charge of basic research for several years and was recently made manager of basic research work in the association laboratories. His present contribution deals with basic considerations for the further study of frost action on concrete (p. 245).

David Watstein and Norman A. Seese, Jr.

collaborated on the study of cracks in reinforced concrete, p. 293.

Mr. Watstein, an ACI Member since 1941, contributed "Bond Stress in Concrete Pull-out Specimens" to the JOURNAL for September 1941 (*Proceedings* V. 38, p. 37) based on his work at the National Bureau of Standards where he had been engaged on several research projects since 1935. (see "Who's Who," N. L. p. 5, Sept. 1941).

Mr. Seese, new to these pages, was graduated from Bridgewater College, Bridgewater, Va. in 1938 with a degree of B.A. in Natural Science. He taught science in high schools of Virginia and West Virginia and also gained one year's experience in surveying prior to going to the Masonry Construction Section of the National Bureau of Standards in May, 1942, where with other members of the staff he made tests of masonry materials and reinforced concrete specimens.

Mr. Seese left the Bureau in Dec. 1944 to enter the U. S. Army and is now receiving his training in Texas.

Clarence Rawhouser

an ACI Member since 1937, who, with R. F. Blanks and H. S. Meissner, contributed "Cracking in Mass Concrete" to ACI *Proceedings* V. 34, p. 477, has worked with the subcommittee of Committee 207 in compiling "Cracking and Temperature Control of Mass Concrete," (p. 273 this JOURNAL). He has been in the dam engineering division of the Bureau of Reclamation in Denver, Colo. since 1930. He has been directly concerned with the problems of temperature control, artificial cooling, and observation of structural behavior of all concrete dams recently constructed by the Bureau, beginning with the pioneering work of that nature on Boulder Dam, and has contributed materially to the plans and methods used for controlling the temperature of Seminoe, Bartlett, Parker, Grand Coulee, Marshall Ford, Shasta, Friant, and Altus Dams. For suggestions made for special temperature-control measures at Friant Dam which permitted elimination of the previously planned longitudinal contraction joints, he received a Department of Interior Award of Excellence.

As a Co-op in the Civil Engineering course of the University of Cincinnati he had varied engineering experience with contracting firms in building construction and with tramway companies in maintenance of track and structures. In 1929 he received the Cincinnati Engineers'

Club cash award to the outstanding senior of the College of Engineering. After graduation he was employed by the U. S. Engineer Office of the War Department in Cincinnati for a year before his appointment with the Bureau of Reclamation. Through resident-extension course work he received the MS degree in Civil Engineering from the University of Colorado in 1943. He is an Associate Member of the American Society of Civil Engineers and a member of Tau Beta Pi.

New Members

The Board of Direction approved 17 applications for membership received in December (12 Individual—3 Corporation —2 Junior) as follows:

Amirikian, A., 6526 Western Ave., Chevy Chase, Md.

Board of Education, City of St. Louis, 911 Locust St., St. Louis 1, Mo.
Attn: Joseph P. Sullivan

Costa, Tercio de Souto, Av. Graca Aranha 206 7° and s. 707/14, Rio de Janeiro, Brazil

Crofts, Capt. W. J., Hq. Royal Aus. Eng., 5th Australian Div., A. I. F., Australia

Davis, Clayton L., c/o Universal Atlas Cement Co., 135 East 42nd St., New York 17, N. Y.

Fett, William E., 914 Washington St., Mishawaka, Ind.

Getman, H. E., 914 Phoenix St., South Haven, Mich.

Hurt, Raymond C., 4592 Roadoan Rd., Cleveland 9, Ohio

Izzett, John, 14483 Linnhurst, Detroit 5, Mich.

James, A. R., Jr., 45 Swan Terrace, Brisbane, Queensland, Australia

Jermann, Arthur Eugenio, Av. Graca Aranha 206 7° and s. 707/14, Rio de Janeiro, Brazil

Milliet, Raul, Av. Graca Aranha 206 7° and s. 707/14, Rio de Janeiro, Brazil

Nicol, James D., Chief Engr., Marwell Construction Co. Ltd., 410 Seymour St., Vancouver, B. C., Canada

Servicos de Engenhari Emilio Baumgart Ltda., Av. Graca Aranha 206 7° and s. 707/14, Rio de Janeiro, Brazil
Attn: Adolfo Pedro Nieckele

Souza, Sergio Valle Marques de, Av. Graca Aranha 206 7° and s. 707/14 Rio de Janeiro, Brazil

Spurr, Henry V., 211-11 33rd Rd., Bayside, L. I., N. Y.

Whan Construction Co. Ltd., Suva, Fiji
Attn: W. Whan

Our 42nd Annual Convention is scheduled for February 18-21, 1946, Hotel New Yorker, New York City

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in 40 annual volumes of ACI Proceedings (since 1929 issued periodically in the Journal of the American Concrete Institute) and in many separate publications.

[To facilitate selective distribution, **separate prints** of this title (41-17) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than July 1, 1945.]

Title 41-17 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 41**

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Vol. 16 No. 5 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **April 1945**

Presidental Address to American Concrete Institute*

By R. W. CRUM

The special committee on postwar planning of the Institute recommended that the "Institute encourage and participate in the discussion of ways and means toward a better postwar world". In its own discussion which followed, the committee assumed that the postwar world would be a peaceful one.

It further recommended that the "American Concrete Institute rededicate itself to the task of increasing, correlating and disseminating the special engineering knowledge of its chosen field . . . " Such an endeavor can only flourish to the advancement of civilization in a peaceful world.

The Concrete Institute is one of many institutions in the fields of science and technology, the sum of whose achievements makes up the advancement in material well being of the peoples of the earth. Basically the activities of these institutions depend upon research, which is nothing but the search for new knowledge, for understanding of it, and for ways to use it for the benefit of man. The whole process was well summed up by Secretary Whipple, who said recently: "As a base of all our work we must, of course, have research to point the way. We do not, however, achieve a large audience except as a few pioneers apply research to practice, and evolve from that practice definite recommendations on what to do about it in design, construction and manufacture."

The material civilization the World enjoys has come about through the summation of such activities of all peoples and institutions in times of peace. In war also, it is true that research flourishes, but it is the research of destruction, and in spite of some war induced discoveries and developments that may have potential value to future civilization, if war conditions prevail long enough, these values may become of little account. For progress, learning must not be lost but enhanced, and for

*By the retiring president, at the Institute's Luncheon Meeting (41st Annual Convention) New York City, Feb. 16, 1945.

learning to increase, scientists and technologists must be trained and embued with the thirst for new knowledge. Even now, who is going to carry the torch after this war is over? Do we realize that already there is a four year gap in the training of scientific and engineering workers that cannot be closed for many years.

This picture is not theory; it is fact, demonstrated many times in the long history of the strivings of the human race toward better life. The civilization of the Roman empire disappeared utterly and was followed by centuries of darkness and distress.

There is no more important duty before the citizens of America today than the effort we must make to secure an enduring peace after this war. To this end the intellectual leadership represented in the membership of this and similar organizations must be exerted to the utmost.

In their essay on "The Problems of Lasting Peace", Herbert Hoover and Hugh Gibson list seven dynamic forces which collectively control international relations with respect to war and peace. Only one of these, "the will to peace", is a positive force in favor of peace as against war. Although this force has been at work throughout the recorded history of civilization it is somewhat discouraging to face the fact that for the most part it has been ineffective. If the "will to peace" were universal throughout the civilized world, the other forces that make for distrust, suspicion and hatred between men and nations could doubtless be overcome but regrettably there have always been men and nations, in whose sight war is good and who do not want lasting peace. Mistaken though they may be, their presence is a fact which renders solution of the problems of lasting peace immensely if not insurmountably difficult.

There have been many proposals for methods of keeping the peace, a few of which have been tried, with indifferent success. The one force at the disposal of government that has been successful in keeping the peace is that of law and law enforcement. It is a fact that in every community, no matter what its size, from a mere village to a group of nations, there is a lawless element that, if unrestrained would keep warfare perpetually active. Civilized nations have learned to maintain internal order through laws, courts and police power. Once, in historical times, international peace was kept by this method. For 300 years imperial Rome governed the civilized world and kept law and order. When its restraining hand fell, for other hundreds of years there was no peace anywhere.

The purpose of this brief essay is to arouse speculation upon the relation of the unescapable facts of war and peace to the prospects for lasting peace, in the years to follow the present worldwide conflict. The fact must be faced that to some nations, to be warlike is regarded as a virtue.

Numerous questions present themselves: Is there any hope that the "will to peace" can be aroused among these peoples? Is there any possibility that through international agreements or coalitions, conditions may be set up that in themselves are conducive to peace and not to war? Are there enough peace-loving nations that will associate themselves together to keep the peace in the civilized world, and if so, will they adopt thorough-going measures to that end? Is there enough wisdom in the human race to protect itself against recurrence of the desperate situation in which it is today?

Each of these questions could be made the basis for an extended essay.

To be more specific: Can enduring peace be assured without the use of police power and if the answer is "no", can enough nations with enough power to enforce the peace be found who will work steadfastly together through this means?

The brief backward look I have taken at history indicates to me that there is little hope for permanent or even long-lasting peace unless a body of just international law is developed, courts established to adjudicate disputes and adequate police power provided. As to the second part of the question, I should say that the record of the human race in developing the arts of peace in spite of wars and pestilences should give rise to well-founded hope that in the end concerted effective action by peace-loving peoples will keep peace between nations just as in any community of civilized individuals, the law-abiding elements work together to uphold the forces of law and order.

The United States of America is foremost among the nations that truly want peace. For many years through our fortunate geographical situation we were able to go our way without embroilment in the quarrels of the rest of the world and in that time we built up the best living conditions yet known to man. Now, through that very research, which under right conditions is our hope for continued upward realization of our aspirations, our safety through isolation has been destroyed. As evidenced by the happenings of this war, another world conflict might entirely destroy our civilization and set us back for centuries.

It is a vital necessity that this nation work wholeheartedly with the other peace-loving peoples to set up a regime of lasting peace, and to my mind that can only be assured through the establishment of international laws, courts and adequate enforcement machinery. I do not believe participation in such a plan for world order need be subversive of our national sovreignity, nor does it need to be incompatible with provision for defense against aggression.

No plan for lasting peace can succeed without the full support of the United States, nor can it succeed without the full support of the powers

allied with us in this war. The problem is indeed difficult, but looking forward to durable world peace must not be dismissed as a mere utopian dream.

[To facilitate selective distribution, **separate prints** of this title (41-18) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than July 1, 1945.]

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Vol. 16 No. 5 7400 SECOND BOULEVARD 2, DETROIT, MICHIGAN **April 1945**

Precast Concrete Pit Sheeting*

By JACOB FELD†
Member American Concrete Institute

SYNOPSIS

The development of light weight concrete slabs in place of wood planks for pit or box sheeting eliminates future settlements of underpinned structures when the buried sheeting rots. Practical use has demonstrated that concrete planks add little to the cost of such work. A summary of the history of box sheeting and of the various types used shows the possibilities for the use of concrete in this phase of construction work.

INTRODUCTION

Excavations for foundation support in restricted localities, such as occur in the digging of pits for the underpinning of buildings, must be performed within sheeted pits with horizontal or box sheeting. Lack of vertical clearance prevents the driving of vertical sheeting. In work where the vibration of the driving would injure adjacent structures or in soils which liquify during such vibration, box sheeting is also a necessary procedure. The underpinning of building foundations along the route of a city subway structure is a major operation, especially when the subway width covers practically the entire street width. In recent years, New York City provisions are that no wood sheeting may be left on the ground, since experience had shown a possible danger of settlements resulting from rotted timber sheeting. In 1935, the writer developed the use of precast concrete box sheeting, used it in the underpinning operations of many buildings and other structures and has found the cost reasonable and only slightly above the most economical wood sheeting.

*Received by the Institute, Dec. 20, 1945.
†Consulting Engineer, New York, N. Y.

PRACTICE AND THEORY

The introduction of horizontal box sheeting is usually attributed to the late James C. Meem, in his work as Chief Engineer for Cranford and McNamee, contractors of the original subway in Brooklyn, N. Y., about 1905. In Cranford and Meem's paper on "Resume of the Development of Underpinning," presented before the Brooklyn Engineers' Club in 1931, he states in connection with the underpinning of Elevated Railway Columns:

> The first important step in connection with this underpinning development was the adoption of a method of sheeting pits, usually not more than 4 x 4 ft. nor greater than 6 x 6 ft. in area, and of any required depth.
>
> The sheeting consisted of 2-in. planks cut so that the pieces on the north and south sides of the pit engaged and braced the square ends of the pieces on the east and west sides, successive courses alternating the bearings from the east and west to the north and south pieces. This was in effect an adaptation and improvement of the method long used by Long Island well diggers, who cut ordinary saplings to the proper length, and placed them in engagement on the opposite sides in the successive courses.
>
> The essential principle in this connection was the fact, well known to some engineers, that it is quite possible to sink separated pits of limited dimensions safely adjacent to a building foundation and to join them by intermediate pits when it is equally impossible to safely excavate there a long continuous trench. This method was, therefore, applied, not only to all the buildings, but to the foundation of the elevated railroad columns.

In a paper on "Pressure and Resistance of Soil," also presented before the Brooklyn Engineers' Club in 1920, Mr. Meem also states his experience that in pits not exceeding 5 ft. square, "the pressure is not cumulative because the horizontal arching component tends to maintain it uniformly, in the same principle, though differently applied, as in grain bins." He also describes the use of a fabricated steel bracing in the form of bolted channel frames placed under each other to act as pit sheeting. This type was quite costly and required considerable labor to remove the sheeting and backfill the pier which was poured within wood forms. Since the pit was 5 ft. square, working room restricted the concrete pier to an uneconomical size.

In 1923, the writer showed that no uncertain internal stress, such as Mr. Meem's horizontal arching component, need be assumed to prove that the lateral pressure of soil on the sides of a pit is not cumulative with depth. In the paper on "Lateral Earth Pressure Determination," published in the 1924 *Transactions*, Am. Soc. of C. E., V. 86, p. 1579, the writer developed a formula for the lateral pressure on the side wall of a pit, based on the modified wedge theory, as follows (see Fig. 1):
Horizontal pressure on a unit area at depth x is

$$w\ x\ tan^2\ \tfrac{1}{2}(90^\circ - \phi) \quad \ldots\ldots (1)$$

where w is the unit weight and ϕ is the angle of internal friction of the soil.

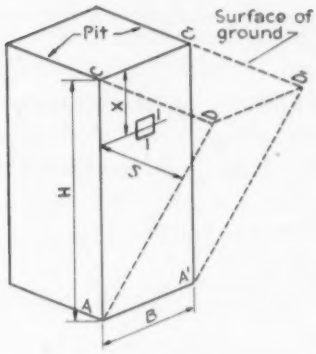

Fig. 1

For an unlimited length of wall, there being no differential motion in the fill, the total horizontal pressure per foot length of wall is

$$E = \tfrac{1}{2}\, w\, H^2\, tan^2\, \tfrac{1}{2}(90° - \phi) \quad \ldots\ldots (2)$$

the usual Coulomb or wedge formula for earth pressure against a vertical wall of a horizontally bounded fill.

If there exists a definite length of wall or sheeting, B, cut out without disturbance, the total pressure on the sheeting equals the pressure on a width B as given by formula (2) less the resistance offered by the undisturbed soil along the vertical planes ACD and $A'C'D'$. It is accurate enough for this purpose to assume that AC is a straight line. The most recent researches indicate that the surface of rupture deviates only slightly from a plane. The tendency is for the wedge of rupture to move parallel to AD; therefore the frictional resistances along planes ACD and $A'C'D'$ are also parallel to AD. In accordance with the wedge theory of earth pressure, angle CAD is $\tfrac{1}{2}(90° - \phi)$.

The total horizontal component of the lateral pressure on area ACD is $\int_o^H w\, x\, S\, tan^2\, \tfrac{1}{2}(90° - \phi)\, dx \quad \ldots\ldots (3)$

where S is the width of the triangle at depth x.

Since $S = (H - x)\, tan\, \tfrac{1}{2}\, (90° - \phi) \quad \ldots\ldots (4)$

then substituting for S and integrating,

$$E\ (\text{on } ACD) = \frac{1}{6}\, w\, H^3\, tan^3\, \tfrac{1}{2}(90° - \phi) \quad \ldots\ldots (5)$$

The resistances along the two planes, since $tan\ \phi$ is the coefficient of internal friction of the soil, are each equal to

E (on ACD) $tan\ \phi$, and the horizontal component of the resistance is E (on ACD) $tan\ \phi\ sin\ \tfrac{1}{2}(90° - \phi) =$

$$\tfrac{1}{6}\, w\, H^3\, tan\ \phi\, tan^3\, \tfrac{1}{2}(90° - \phi)\, sin\, \tfrac{1}{2}(90° - \phi) \quad \ldots\ldots (6)$$

For horizontal equilibrium, i.e., no increase in total horizontal pressure, the total pressure given by formula (2) over the width B, must be bal-

anced by the two resistances, each given by formula (6). Such equilibrium will occur when

$$\frac{B}{H} = \frac{2}{3} \tan \phi \tan \tfrac{1}{2}(90° - \phi) \sin \tfrac{1}{2}(90° - \phi) \quad \ldots\ldots\ldots\ldots\ldots (7)$$

From this formula, Table 1 has been computed, which gives for various values of ϕ,

Column (*c*) the ratio of B/H below which there is no increase in cumulative lateral pressure.

Column (*d*) the depth H_o beyond which there is no pressure in a pit 5 ft. wide.

It will be noted that H_o is a minimum for values of ϕ about 35°. The values in Table 1 indicate depths where the net total pressure is zero, i.e., the surplus resistance in the lower levels neutralizes the unbalanced part of the pressure at the upper levels. This state of affairs, most marked in soils of ϕ about 35° (the usual type encountered) explains the often reported observation that the pressure in pits and in rigidly

TABLE 1

ϕ Angle of Friction (deg.)	(a) Values of B/H for Zero Unit Pres.	(b) Values of B/H for Max. Unit Pres.	(c) Values of B/H for Zero Ttl. Pres.	(d) Depths in Pits 5 ft. Wide at which Total Pres. is Zero	(f) Depths in Pits 5 ft. Wide at which Unit Pres. is Max.	(g) Depths in Pits 5 ft. Wide at which Unit Pres. is Zero	(e) Max. Pres. on 5 ft. wide by 1 ft. deep (lb.) for W = 100 lb./c.ft.
0	.0	.0	.0	Infinite	Infinite	Infinite	Infinite
5	.054	.108	.036	139	46	93	19300
10	.098	.196	.065	77	26	51	9100
15	.126	.252	.084	60	20	40	6000
20	.147	.294	.098	51	17	34	4200
25	.159	.318	.106	47	16	31	3700
30	.167	.334	.111	45	15	30	2500
35	.168	.336	.112	45	15	30	2000
40	.165	.330	.110	45	15	30	1650
45	.159	.318	.106	47	16	31	1360
50	.149	.298	.099	51	17	34	1100
55	.135	.270	.090	56	19	37	950
60	.120	.240	.080	62	21	42	740
65	.104	.208	.069	72	24	48	600
70	.084	.168	.056	89	30	60	450
75	.065	.130	.043	116	39	77	390
80	.042	.084	.028	178	60	119	300
85	.024	.048	.016	435	104	208	260

sheeted excavations, seems to decrease with depth, is zero at the bottom of the excavation and maximum at the top or near the top.

For the design of horizontal sheeting, we are more interested in unit pressures at various depths than in total pressures, and especially in the value of maximum unit pressure.

The net pressure on a side of the pit, of width B, from equations (2) and (6) becomes

$$E(on\ ACC'A') = \tfrac{1}{2}\, w\, BH^2\, tan^2\, \tfrac{1}{2}(90 - \phi) - \frac{1}{3}\, w\, H^3\, tan\, \phi\, tan^3\, \tfrac{1}{2}(90 - \phi)\, sin\, \tfrac{1}{2}(90 - \phi)$$

$$= w\, H^2\, tan^2\, \tfrac{1}{2}(90 - \phi) \left(\frac{B}{2} - \frac{HT}{3}\right) \ldots\ldots\ldots (8)$$

Where $T = tan\, \phi\, tan\, \tfrac{1}{2}(90 - \phi)\, sin\, \tfrac{1}{2}(90 - \phi)$.

The net pressure on a strip one foot high, B wide and at depth H is

$$E_H = w\, tan^2\, \tfrac{1}{2}(90 - \phi)\, (BH - H^2T) \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

E_H is maximum when $\dfrac{\partial E_H}{\partial H} = O = B - 2\, HT$

or when $B = 2\, HT \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$

or $H = \dfrac{B}{2T}$

$$E_H \text{ maximum} = w\, tan^2\, \tfrac{1}{2}(90 - \phi)\, \frac{B^2}{4T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Table 1 gives values for maximum unit pressure (e) and depths at which maximum (f) and zero unit pressures (g) occur in pits 5 ft. wide.

It will be noticed that for the usual soils (values of ϕ between 15 and 60 degrees), the maximum unit pressure occurs within a depth of 20 ft. Pits sunk to any depth need sheeting to sustain only that maximum pressure. This explains such observations as for instance, the article by H. F. Peckworth on "Sheeting for Underpinning Pits" in *Civil Engineering* July 1934, page 367, that 6 ft. by 6 ft. pits were sunk through damp sand 65 ft. without the slightest difficulty, using 2 x 8 timber sheeting.

PRACTICAL CONSIDERATIONS

The simplest pit sheeting consists of square edged boards, successively placed one sheet at a time below the previously braced sheets (Fig. 2a). This necessitates careful excavation, by hand, cutting the soil to closely simulate the shape of the sheet to be placed. Where soil conditions do not permit such shaping, and where the soil will not remain with a vertical face until the next sheet is placed and wedged, any one of several "louvre" type sheets are used. The basic principal of louvre sheeting is

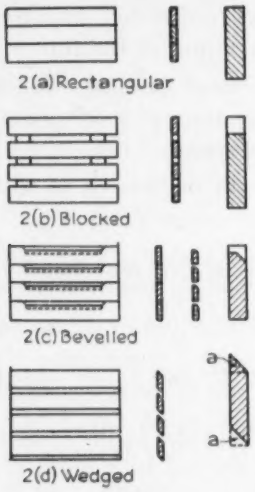

Fig. 2—Elevations, sections, details of types of box sheeting

the provision for packing in back of the sheets, to correct for irregularities of the excavation and to eliminate open voids in back of the sheeting.

Several types of louvre sheeting have been developed. The earliest (Fig. 2b) design was to interpose short blocks in gaps between rectangular sheets. Working space in a pit is limited and the fewer items to handle and place, the more work can be accomplished. As a result, the blocks are often omitted, to be placed after the pit is fully excavated.

To avoid the use of the small blocks, a patented beveled sheet has been developed (Fig. 2c) which provides continuous support of the pit sheeting with a gap for packing. Incidentally, these gaps are often packed with straw or hay, in fine water bearing soils, to stop the flow of the soil and to drain out free water. To reduce the cost of the sheeting and at the same time providing a sloping surface in the gaps between sheets, which makes it easier to pack and also reduces the possible flow of soil into the pit, the late Jacob Siebert, formerly Superintendent on numerous New York City subway contracts, developed the type shown in Fig. 2d. An 8-in. board covers 10 in. of vertical exposure. The sharpened lower edges of the boards permit digging into the bottom of the pit and rotating the board into position against the side of the excavation.

Bracing of the pit sides can be accomplished in several ways, all of which use the principle that the completed pit is in equilibrium and two opposite sides will support the pressure on the other two sides. All of the methods used in making wooden boxes are employed on a larger scale in pits. Several methods are shown in Fig. 3.

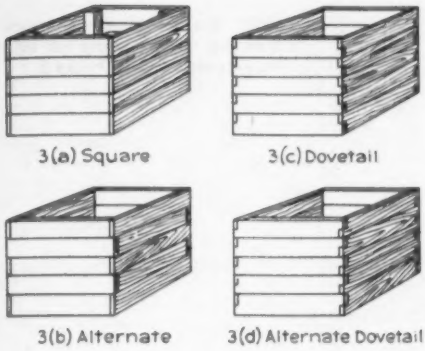

Fig. 3—Types of box sheeting

Where pits are sunk for underpinning and the sectional construction of a continuous wall, three sides of the pit sheeting are usually removed, after the pit has been concreted. The fourth side cannot be removed since it is under the building which is being underpinned. In the 1935 specifications of the N. Y. City Board of Transportation, a definite prohibition against the use of permanent wood sheeting was not included, but before construction started in 1936 on the Sixth Avenue Subway, the order was received by the Contractor that no wood sheeting could remain in the ground. The object was to eliminate future rotting of the timber and possible later consolidation and settlement of structures.

The writer, then Chief Engineer for the Brader Construction Corp., in charge of the subway section in 6th Avenue between 39 and 46th Street, the first section contracted, made a study of precast concrete slabs as a substitution for timber in underpinning pits. The usual and customary opposition to anything new, came from the field forces. However, by placing engineering assistants in direct charge of the work in the field, such inertia was soon overcome, and the underpinning foremen soon made public at every possible occasion, their experience in the use of "non-wood" pit sheeting. Some years earlier, the writer had designed, supervised the manufacture and installation and had tested thin precast concrete slabs used as the seat deck of the Rochester (N. Y.) Ball Park grandstand. It was natural to use such prior experience and the results of those tests were used to prove the feasibility of concrete slabs as sheeting.

Since there were about a hundred buildings to be underpinned in the Brader contract, pits expected from 5 to 40 ft. in depth, economy of construction was a controlling item. Several types of units were studied and designed and those which looked possible were tabulated and fabrication prices obtained from local concrete product manufacturers. Light weight concrete was preferred, since the work is entirely manual and in the cramped pit, handling planks is tiring labor. The result of

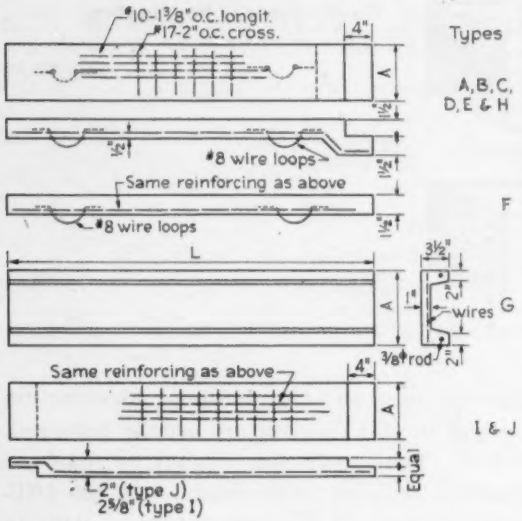

Fig. 4(a)—Precast concrete sheeting slabs—study for subway contract (see Table 2.)

TABLE 2—DETAILS AND COST OF PROPOSED TYPES

Type	A	L	Wt. lb. per Sq. ft.	Aggregate	Ultimate Load per Sq. Ft.	Cost per Sq. Ft.
A	8"	60"	18½	Sand	200	$0.20
B	12	60	"	"	"	.20
C	8	60	14	Haydite	"	.22
D	12	60	"	"	"	.22
* E	8	48	"	"	"	.22
* F	8	56	"	"	"	.22
G	24	96	12	Porete	240	.215
H	12	60	13	Haydite	200	.42
I	12	60	16	Porete	"	.36
J	12	60	17	Haydite	"	.32

Costs are bid prices F.O.B. job in truckload lots (7½ tons)
Add $0.01 per sq. ft. for Incor cement

*Types used.

the price canvas is reproduced in Fig. 4, which also shows the typical bracing method and the steps of the underpinning operation.

The simplest type was chosen and found entirely satisfactory. The planks were 8 in. wide and ordered in two lengths since flexibility was needed in pit widths. Each building required a separate underpinning to avoid property ownership encroachments. Pier widths were determined for each building in combinations of 48- and 56-in. lengths. It was felt that 8 in. was sufficient height, since the sheeting has to be undercut several inches more than the height of the sheet being placed. To avoid placing the sheets with reinforcing in the wrong face, two wire loops protruded from the front face. These loops served as hand grips

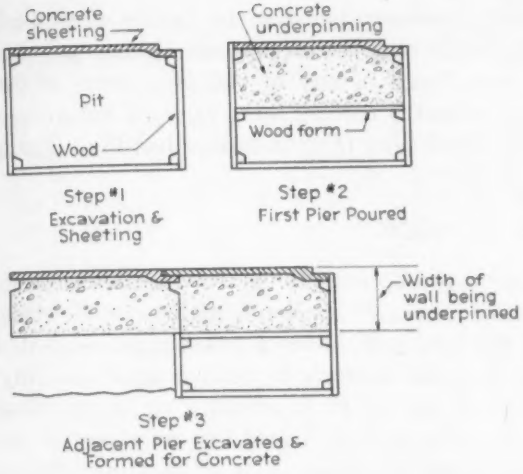

Fig. 4(b)—Use of sheeting

for lowering and placing the sheets and also a second purpose for hanging the sheet as it is placed and until it is braced in position.

The sheets weighed less than 10 lb. per foot of length and no objections were raised by the laborers. As a matter of fact, the units looked much heavier than their 40 lb. weight and men expressed surprise at how easy it was to pick up and place the planks.

TESTS OF SHEETING

Before installing the first planks, a load test was run on six slabs by two quarter point loadings with a 44-in. span. The slabs were loaded to failure in a Riehle testing machine, three at an age of seven days and three at 28 days after manufacture.

Tested slabs were 8 x 48 x 1½ in. (test span 44 in.) reinforced with No. 10 AS & W wires, 1⅜ in. o.c. longitudinal and No. 17 AS & W wires 2 in. o.c. crosswise, placed ½ in. from face of slab. Haydite aggregate and Incor portland cement were used by the Federal American Cement Tile Co. in manufacturing these first slabs.

In the 7-day age test, the first crack appeared at a minimum total load of 611 lb., with a deflection of 0.7 in., equivalent to a uniformly distributed load of 258 lb. per sq. ft. As additional load was added, more cracks opened up, but the steel did not fail in tension and at a load of 670 lb., the deflection was 2½ in.

In the 28-day age test, the first crack appeared at a minimum total load of 625 lb., equivalent to a load of 264 lb. per sq. ft. All specimens tested showed strengths within 5 percent of the values noted above, which convinced everyone that proper control was being exercised by

the manufacturer. This was important, because the failure of a plank in a pit would almost always result in a serious personal injury, possibly a fatal one. No failures were found in any of the pits; some 20,000 planks were used. However, about 5 percent were rejected before use, most often because of cracks developing from improper handling during delivery.

COSTS

After using these units for several weeks, a careful check was made of labor costs compared to the previous experience with wood sheeting. The reports from several of the field staff, showed a maximum estimated time loss of 10 minutes per sheet for concrete as against wood sheeting. This corresponds to about $0.14 per sq. ft. of sheeting or about $0.40 per cu. yd. of pit volume. To this must be added the extra cost of the concrete sheeting over timber, about $0.18 per sq. ft. of sheeting, equivalent to about $0.50 per cu. yd. of pit volume. This adds about $1.00 total per cu. yd. to an average cost of $27.00 to $40.00 per cu. yd. for the completed underpinning (including the concrete filling of the pits and the wedging up of the loadings). The cost is based on labor prices of 1939 in the New York area.

Progress of excavation including placing of the sheeting averages about a foot per hour, so that the maximum loss in progress by using concrete sheeting would not exceed 15 percent and probably would be practically zero.

The field studies indicated that the extra time resulted from the greater flexibility of wooden pit boards. When the excavation is not "full" and the fit is tight, a wood plank can be driven into place by means of a mall, the concrete plank cannot be treated in this manner, for fear of shattering or cracking it. It was therefore seen to it that the excavation was carefully done to proper fit. Also when boulders were encountered, the wood plank was cut out with an axe to fit around the boulder, but with concrete plank, the boulder had to be drilled and cut off to make room for the boards. However, the concrete pit board is a permanent material which will not be affected by dry rot or water level fluctuations.

More recently, when the problem arose of building a retaining wall to safeguard an important watermain during the adjacent excavation for a reservoir, it was considered essential to use a permanent pit sheeting to guarantee against possible future loss of lateral support. Fig. 5 shows the plan and section of the bracing piers installed. By inserting sheeting in the gaps between piers, full protection was provided by building only a 50 percent continuous wall. Protection over a length

of 160 ft. (16 piers) cost approximately $6400. The work was done in 1942 in a mid-eastern city, with almost ideal digging materials.

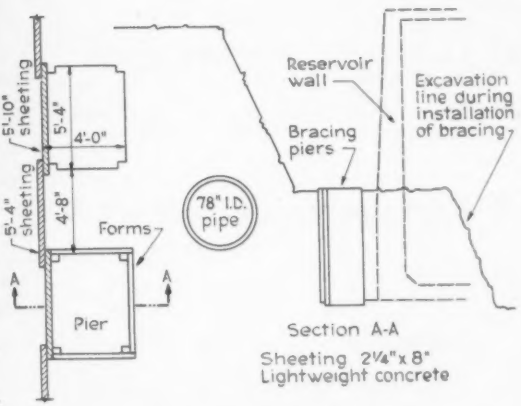

Fig. 5—Precast concrete sheeting in permanent protection wall

Fig. 6 (below) — Sheeting in the pit

Photographs of underground work are seldom available to illustrate the technique of new operations. However, Fig. 6 (taken by the author) looking down into a pit under the foundation of a building being underpinned shows a succession of pit boards tied up by wire through the wire loops. The planks are 56 in. long and 8 in. high.

[To facilitate selective distribution, **separate prints** of this title (41-19) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than July 1, 1945.]

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A Practical Procedure for Rigid Frame Design*

By D. R. CERVINt
Member American Concrete Institute

SYNOPSIS

Moment distribution has appreciably simplified theoretical studies of rigid building frames but in itself is not always a practical office tool. Two-cycle moment distribution coupled with short-cuts in loading for maxima has reduced the time element in design procedure considerably but still leaves something to be desired for actual office usage. This paper attempts to carry the two-cycle method one step further, illustrating a procedure whereby any rigid frame can be completely designed within a time period that is economically feasible for average office usage.

SYMBOLS AND NOTATIONS

a : coefficient used in $A_s = \frac{M}{ad}$
A : gross area of columns
A_s : area of tensile reinforcement; also area of web reinforcement
b : width of beams or columns
c : depth of column in direction of bending
C : $\frac{f_a}{0.45f'_c}$
d : effective depth of beams
D : $\frac{t^3}{2R^2}$; used in design of columns having bending moments
DF_{bm} : distribution factor for beams
DF_{col} : distribution factor for columns
DL : dead load
E_c : modulus of elasticity of concrete
E_s : modulus of elasticity of steel
f_a : average allowable stress on axially loaded column
f_c : compressive stress in extreme fiber
f'_c : ultimate compressive strength of concrete
f_s : stress in tensile reinforcement
f_v : stress in web reinforcement
FEM_D : fixed end moment due to dead load
FEM_T : fixed end moment due to total load
FM : final moment
g : distance (gc) between bars at opposite faces of column to overall dimension (c)
h : length of columns (floor to floor)
j : ratio of distance (jd) between resultants of compressive and tensile stresses to effective depth
k : ratio of distance (kd) between extreme fiber and neutral axis to effective depth
K : $\frac{1}{2}f_c jk$; also relative stiffness of beams and columns

*Received by the Institute Dec. 8, 1944.
†Structural Engineer, National Advisory Committee for Aeronautics, Langley Field, Virginia.

l : centerline distance between columns; also span supported by columns
LL : live load
M : external moment
Mid_D : moment at mid-span for dead load due to fixity at ends
Mid_T : moment at mid-span for total load due to fixity at ends
n : ratio of $\frac{E_c}{E_s}$
N : external concentric load on columns
p : ratio of longitudinal reinforcement in columns
P : equivalent total concentric load on columns
s : spacing of concrete frame
S : base length of shear diagram
TL : total load
v : shearing stress
v' : shearing stress taken by web reinforcement
V : total shear
w_D : uniformly distributed dead load
w_T : uniformly distributed total load

1. PROBLEM

The introduction of moment distribution* to the engineering world during the past decade was hailed by many as marking a new era in rigid frame design and analysis. No one can deny that this has taken place. The mechanical procedure of moment distribution is so easy to learn and to apply that many problems bordering on "higher engineering" in complexity are now within the capabilities of average, practicing engineers. Surely in the field of theoretical studies and analyses, great masses of engineers are now capable of fully understanding and intelligently discussing problems which two decades ago were the chief property of professors and advanced graduate students of engineering.

All fields of engineering, however, have not benefited equally. Thus, in the design of reinforced concrete building frames where loading for maxima are required, the use of moment distribution makes the problem theoretically practical when compared to the older classical methods; but so much time is required to determine moments for the some twenty loading cases necessary for an average small rigid frame as to render this method economically impractical for general office usage. Instances can be cited where some offices have vastly reduced this voluminous amount of work by employing the simple expedient of simultaneously loading all spans with full live load and designing the frame for the resulting moments. This is, of course, not only contrary to the requirements of any recognized building code, but is also contrary to the concepts of any thinking engineer.

The use of two-cycle moment distribution combined with short-cuts in application and loading arrangements for finding maximum moments† has certainly placed the problem of rigid building frame design within the grasp of average engineers. Yet a step by step procedure of a frame three stories high and four bays wide requires too much time to be acceptable to some offices. Certainly many offices have accepted it; but for each office or engineer that uses this method, one can point to

*See "Continuity in Building Frames" by Cross and Morgan. Also PCA ST-40, 41, 42 and 43.
†See PCA "Continuity in Concrete Building Frames", Third Edition.

another group that has rejected it and returned to archaic, uneconomical, and sometimes "code-contrary" arbitrary moment coefficients.

To that group of engineers which still believes that an "exact" method of rigid frame design is too time-consuming for economical office usage, the following arrangement of the two-cycle method is offered.

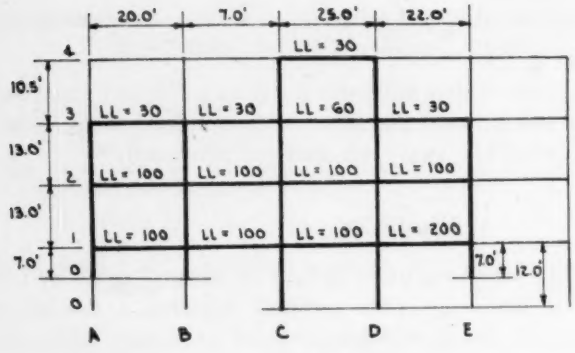
2. SOLUTION

Most concrete engineers were originally taught to design beams by use of arbitrary moment coefficients. This method requires a minimum number of sketches, most of the time being devoted to routine slide-rule computations. When trying moment distribution, once its novelty wears off the designer cannot keep from noting the vast amount of time necessary to make the numerous sketches required to prepare a predesign of the frame from which relative stiffnesses and distribution factors for beams and columns can be computed, and to calculate *FEM*s. Until the time required to perform these operations is shortened, many designers are prone to stay away from any form of moment distribution for practical office usage.

A partial solution immediately suggests itself. The preparation of blank forms applicable to any reasonably sized frame is a step in the right direction. These forms can be penciled on tracing paper, inexpensively reproduced, and then filed away ready for instant use.

Again referring to the normal design of beams by arbitrary moment coefficients, the usual procedure is to completely design one member at a time, starting with dimensions and loads and winding up with size and spacing of stirrups. Many more calculations are necessary when moment distribution is used and a more efficient layout procedure must be adopted. It is apparent that if similar operations on all beams and columns are performed successively, rather than each member designed completely as a unit, a more efficient use of time will result. Another important advantage accrues from this procedure. The mass repetition of a single operation lends itself well to columnar layout on standard form sheets. It also has the further advantage of permitting easier checking.

These two time-savers, prepared blank forms and immediate repetition of similar operations, are the basis of this paper. Admittedly this procedure still takes more time than that required by arbitrary moment coefficients, but after one or two complete designs, the time necessary is well within any reasonable office allowance. Broken down to its elements, this method is so simple that it can be used successfully by anyone who has just a fair understanding of moment distribution; and



SPECIFICATIONS

f_s = 24000 psi
f'_c = 3000 psi
f_c = 1200 psi
n = 10
f_v = 60 psi

For 24000/10/1200:

K = 178
k = 0.333
j = 0.889
a = 1.76

Beam	Slb&Jst	Ceil	Roof	Girder	Partition	Fl. Fin.				Total DL	LL	TL
4 CD	50	5	10	20	-	-				85	30	115
3 AB	50	10	10	20	-	-				90	30	120
3 BC	40	10	10	20	-	-				80	30	110
3 CD	60	10	-	20	20	5				115	60	175
3 DE	50	10	10	20	-	-				90	30	120
2 AB	70	10	-	25	20	5				130	100	230
2 BC	40	10	-	20	-	10				80	100	180
2 CD	70	10	-	25	20	5				130	100	230
2 DE	90	10	-	35	-	5				140	100	240
1 AB	70	-	-	25	20	5				120	100	220
1 BC	40	-	-	20	-	10				70	100	170
1 CD	70	-	-	25	20	5				120	100	220
1 DE	90	-	-	35	-	-				125	200	325

Sheet 1—Frame layout and summary of loads

it is particularly suited to larger offices where definite design standards must be adhered to.

3. EXAMPLE

The procedure is best illustrated by carrying through the complete design of an actual building frame. Eleven sheets are necessary. The first nine comprise the design calculations, while the last two are to be added to the contract drawings. Comment and description of these sheets follow.

Sheet 1. Frame layout, and summary of loads

The frame selected has considerable variation in both span and liveloading. Since the second span is flanked by spans of considerably greater length, large negative moments at midspan can be expected. Note that all the data on this sheet, as well as on all other sheets, are placed on prepared blank forms.

①	②	③	④	⑤	⑥	⑦	Uniform ⑧	Uniform ⑨	Uniform ⑩	Uniform ⑪	⑫	⑬	⑭	⑮
Beam	DL	TL	S (width)	w_D ② ④ (kips)	w_T ③ ④ (kips)	ℓ (length)	FEM_D Table II	FEM_T Table II	Mid_D $\frac{1}{2}$ ⑧	Mid_T $\frac{1}{2}$ ⑨	b (beam width)	d $\sqrt{\frac{⑨\ 12}{K\ ⑫}}$	d Adjusted	2K Table III
4CD	85	115	19	1.6	2.2	25	83	114	42	57	12	25	25	125
3AB	90	120	19	1.7	2.3	20	57	77	28	38	12	21	31	298
3BC	80	110	19	1.5	2.1	7	6	9	3	4	12	7	31	853
3CD	115	175	19	2.2	3.3	25	114	172	57	86	12	31	31	238
3DE	90	120	19	1.7	2.3	22	69	93	34	46	12	23	31	271
2AB	130	230	19	2.5	4.4	20	83	146	42	73	14	27	33	420
2BC	80	180	19	1.5	3.4	7	6	14	3	7	14	8	33	1200
2CD	130	230	19	2.5	4.4	25	130	229	65	114	14	33	33	336
2DE	140	240	19	2.7	4.6	22	109	185	55	93	14	30	33	382
1AB	120	220	19	2.3	4.2	20	77	140	38	70	16	24	24	184
1BC	70	170	19	1.3	3.2	7	5	13	3	7	16	7	24	527
1CD	120	220	19	2.3	4.2	25	120	218	60	109	16	30	33	384
1DE	125	325	19	2.4	6.2	22	97	250	48	125	16	33	33	436

Sheet 2—FEM, Mid Mom, and K-values for beams

(1) Col.	(2) h (height)	(3) W_T	(4) ℓ (length)	(5) TL ③④	(6) Outside Wall	(7) Conc. Load	(8) Weight Col.	(9) N Σ ⑤⑥⑦⑧	(10) ΣN	(11) A $\frac{\Sigma N}{0.9}$	(12) b	(13) $c = \frac{A}{b}$ min = 10"	(14) K Table IV
A32	13.0	2.3	10.0	23	8		2	33	33	36	12	10	8
A21	13.0	4.4	10.0	44	25		3	72	105	117	14	10	9
A10	7.0	4.2	10.0	42	25		3	70	175	194	16	12	33
B32	13.0	2.2	13.5	30	-		2	32	32	36	12	10	8
B21	13.0	3.9	13.5	53	-		3	56	88	97	14	10	9
B10	7.0	3.7	13.5	50	-		3	53	141	158	16	10	19
C43	10.5	2.2	12.5	28	8		2	38	38	42	12	10	10
C32	13.0	2.7	16.0	43	20		2	65	103	115	12	10	8
C21	13.0	3.9	16.0	62	-		3	65	168	187	14	14	25
C10	12.0	3.7	16.0	59	-		3	62	230	256	16	16	45
D43	10.5	2.2	12.5	28	8		2	38	38	42	12	10	10
D32	13.0	2.8	23.5	66	20		2	88	126	139	12	12	13
D21	13.0	4.5	23.5	106	-		3	109	235	260	14	20	73
D10	12.0	5.2	23.5	122	-		3	125	359	400	16	24	155
E32	13.0	2.3	11.0	25	8		2	35	35	39	12	10	8
E21	13.0	4.6	11.0	51	25		3	79	114	127	14	10	9
E10	7.0	6.2	11.0	68	25		3	96	210	234	16	16	77

Sheet 3—Axial load and K-values for columns

Sheet 2. FEM, mid moment and K-values for beams

The *FEM*'s and midmoments shown in cols. 8, 9, 10, and 11 are taken from Table 2 in "Continuity in Concrete Building Frames". Those who object to using tables can compute moments by employing conventional slide rule procedure. In computing relative stiffnesses, note that $2K$ is used instead of K, the doubled value being used to more nearly approximate the T-action to which these beams are subjected.

In sizing the structure for computing distribution coefficients, the designer must decide whether a uniform depth or variable (for balanced design) depth of beam for any one floor should be adopted. The former practice will simplify form construction and erection at the cost of some extra concrete. For either method, a trial depth is calculated (col. 12) based on the *FEM* (col. 9) and that beam width which will produce a depth not interfering with any headroom requirements. If a uniform beam depth per floor is adopted, the greatest trial depth (col. 13) should be used. Many test cases have verified the reasonable accuracy of this procedure. When a variable depth is to be used, the problem is considerably complicated by the many combinations of lengths of spans and loadings. As a general rule, the maximum trial depth will change negligibly, while the lesser trial depths increase in size. In this frame, constant depths are to be used for floors 2 and 3, while floor 1 is divided into two groups of two beams each, the depth of each group being determined by the maximum trial depth in that group. Final computed depths (Sheet 7) indicate that sizing of the members is correct for all practical purposes.

The design illustrated considers uniform loads only. If concentrated loads exist, additional columns for concentrated *FEM*s must be inserted between cols. 11 and 12.

Sheet 3. Axial load and K-values for columns

To shorten the number of calculations, the running load per foot of supported floor (col. 3) is the average of the two supported beams (Sheet 2, col. 6). In sizing the columns, an average compressive strength of 900 psi is used to compute the required area. The relative stiffnesses of the columns K, can be either taken from Table IV or computed in conventional slide rule manner.

Sheet 4. K-values and distribution factors at joints

No comment.

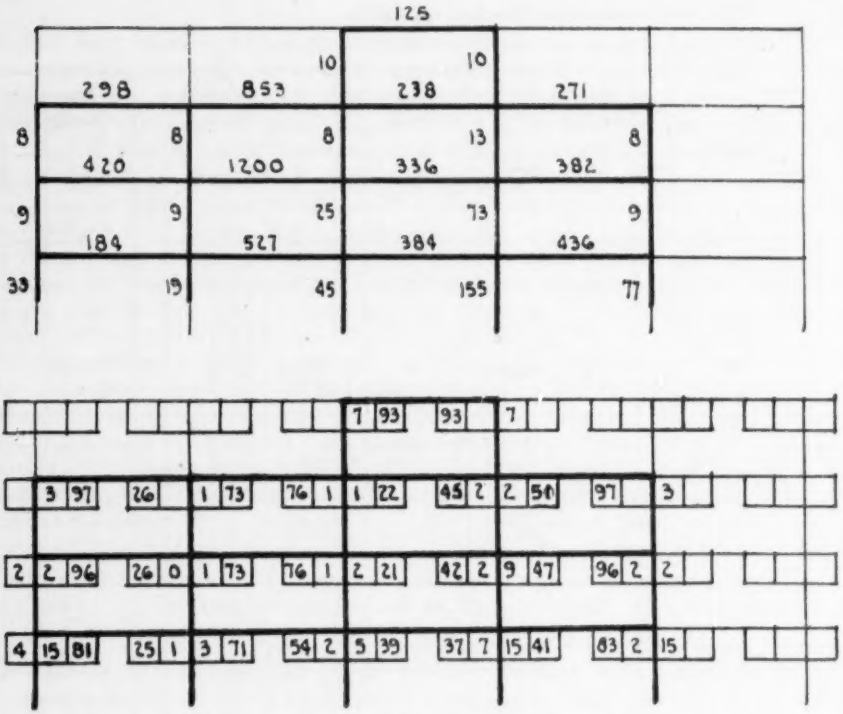
Sheets 5 and 6. Maximum positive, negative, and exterior column moments. Minimum positive and maximum interior column moments

The distribution process follows exactly the two-cycle method outlined in "Continuity in Concrete Building Frames." It is a safe conjecture to make that if the normal process of moment distribution were applied to this frame, coupled with the multiplicity of loadings necessary to produce maximums, the work entailed would have been increased four- or five-fold.

Sheet 7. Design of beams

Since both the Joint Committee report (1940) and ACI Code* require that moments be computed by centerline distances but permit beams to be designed by moments at faces of supports, a correction of the moments already computed may be made. This correction is made by deducting $\frac{1}{3}Vc$ at the supports and $\frac{1}{6}Vc$ at the centerline, where V equals simple shear and c equals the depth of the column. For minimum moments, the correction is added rather than deducted. A complete discussion of this point can be found in "Continuity in Concrete Building Frames."

*Building Regulations for Reinforced Concrete" (ACI 318-41)

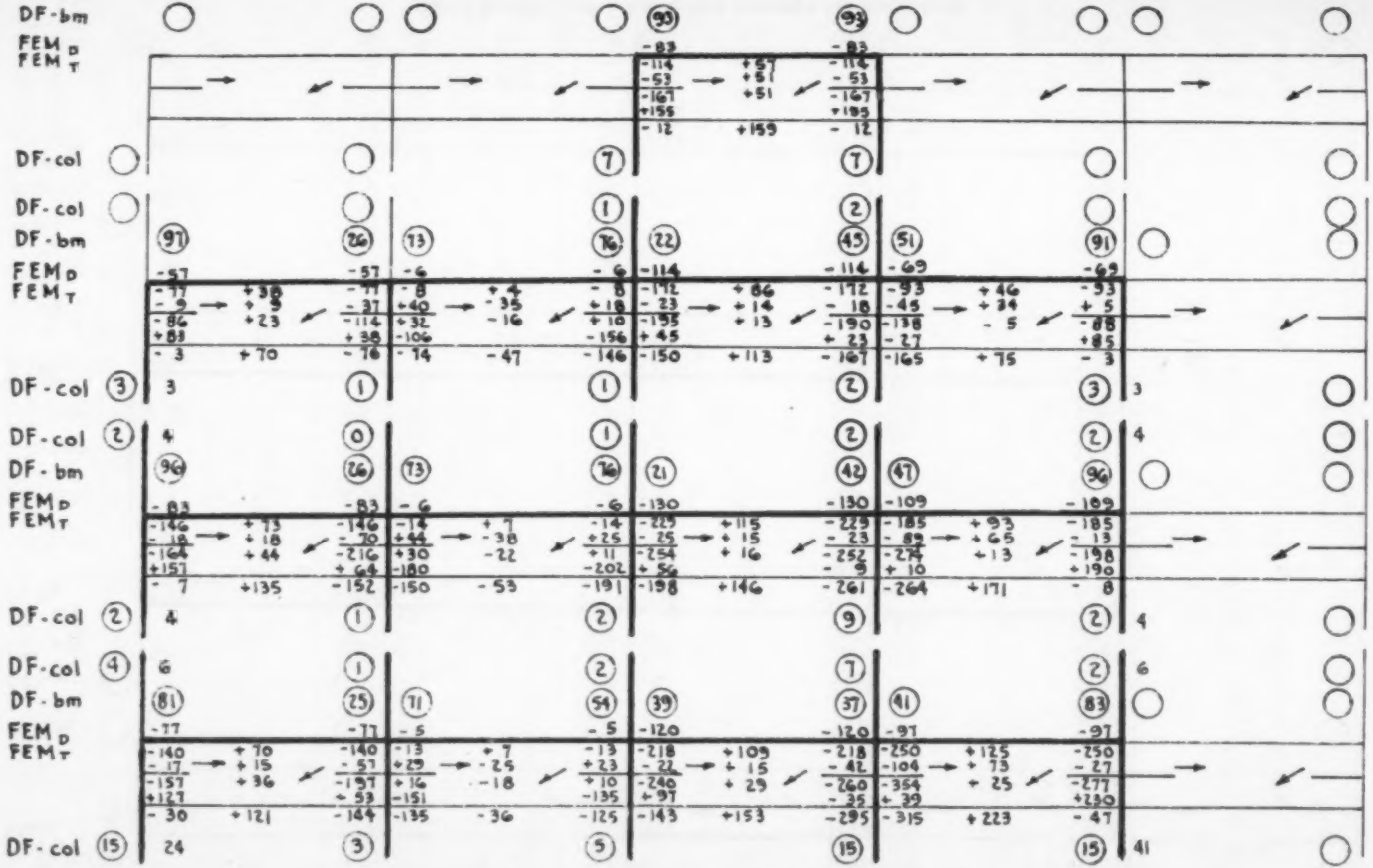


Sheet 4—K-values and distribution factors at joints

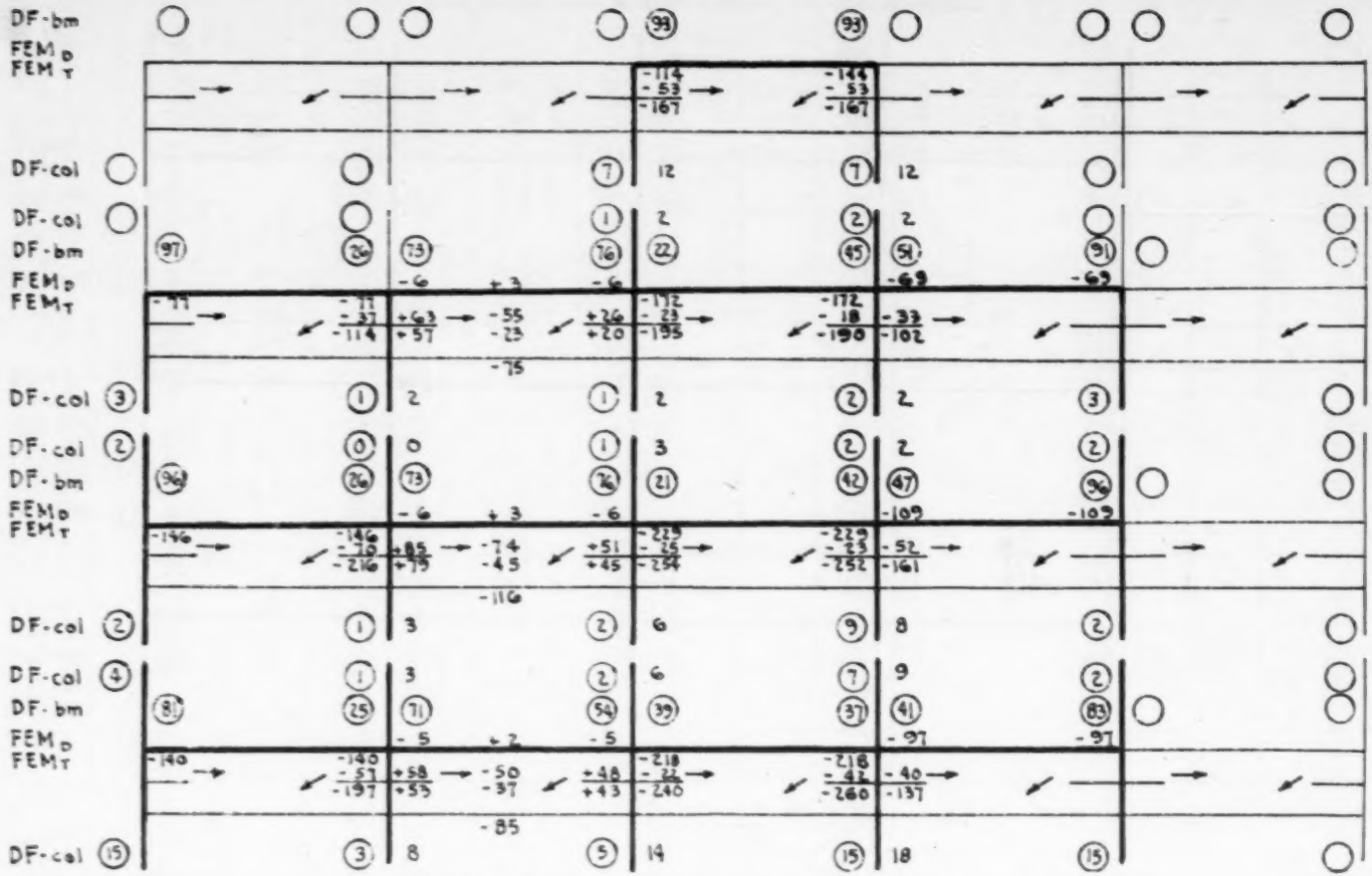
A comparison of the final required beam depths with the preliminary depths (Sheet 2) is very favorable for floors 2 and 3. For floor 4 (upper Roof level) the estimated depth of 25 inches is 4 inches less than the required depth of 29 inches. This increased depth will also increase the stiffness of the beam, but since it is already taking 93 per cent of the unbalanced moment at the corner joints, any change will be relatively small. However, the head room of this already low ceiling room has been reduced another four inches. It is therefore necessary to increase the 10.5 foot distance between levels 3 and 4 to 11.0 feet, or design the beam with compressive reinforcement.

At floor 1, beams 1-AB and 1-BC check out exactly, but the estimated 33-inch depth for 1-CD and 1-DE must be increased to 36 in. Revised moments computed on the increased stiffness of this deeper beam result in no moment increase greater than several per cent. This is shown on Check Sheet 1 for which beam stiffnesses based on a depth of 36-in. are used. This demonstration is included to show that final depths can vary appreciably from preliminary depths with only negligible changes in the moments. Such a check may usually be omitted.

Inflection points are computed to the nearest half-foot by determining the ratio of final moments to *FEMs* and entering this figure on the diagram shown. This diagram assumes symmetrical end moments but is reasonably accurate for unsymmetrical conditions. However, where one end moment is close to zero, appreciable errors may result from the use of this diagram. A study of inflection points for beams having zero moments at one end reveals that coefficients for finding the inflection points at the opposite ends are exactly equal to one-sixth of the ratio of final moments to *FEMs*.



Sheet 5—Maximum positive, negative and exterior column



Sheet 6—Minimum positive and maximum interior column

All end spans for this frame have negligible moments at the wall end, and this rule for locating inflection points at the opposite end is used.

Note that in designing the steel for beams, the coefficient a is used in lieu of $\frac{f_s}{12000}$ x (average j value). This small time saver per beam represents a substantial saving in time for all 13 beams.

Sheet 8. Design of columns

Accurate column design involving both bending and axial load is usually a fairly time-consuming problem. The method illustrated on sheet 8 is accurate within a very small per cent and requires little more time than for columns having direct load only. In column 5, the value of CD is assumed equal to 3 for tied columns, 4 for square spiral columns, and 6 for round spiral columns. Thus the equivalent axial load can be immediately found and the column is designed by Table 18 in the ACI Reinforced" Concrete Design Handbook".*

Some question might be raised regarding the accuracy of the above assumed values of CD. To prove its accuracy, consider a redesign of Column C-21 by the method outlined in "Continuity in Concrete Building Frames."

Refer to Table VII.

For $g = 0.71$, $(n-1)p = 0.20$ (estimated) and rectangular section with ties: D = 5.6

Refer to Table VI.

For $f'_c = 3000$, $f_s = 20{,}000$, $p = 0.020$ (estimated) and tied columns: $C = 0.54$.

Compute $CD\frac{M}{e} = 0.54 \times 5.6 \times \frac{6}{14} \times 12 = 16$

Add	$N = 168$	
Design section for total load	$P = 184$	About equal
By short-cut above	$P = 183$	

Sheet 9. Design of stirrups

Since continuity is an essential factor in this design, it will be necessary to correct shears due to unequal end moments. The shear correction for end spans can be computed directly from end moments shown on Sheet 5. For interior spans, however, the smaller of the two end moments for any beam can always be reduced by a proper reloading of the frame, thereby increasing the maximum end shear. A study of many interior beams reveals that the shear correction is rarely more than double the value obtained by using the end moments shown on Sheet 5. Since the shear corrections due to continuity is usually only a small per cent of the simple shear, the doubling method just outlined is a rapid, safe, and reasonably economical way to achieve an accurate result.

In computing shears, note that the simple shear (col. 5) has already been computed on Sheet 7. Once the corrected shears have been calculated, the method of stirrup design is modeled directly after the procedure outlined in "Reinforced Concrete Design Handbook."

Sheets 10 and 11. Beam and column schedules

These two sheets compile the actual design requirements into the usual tables which appear on most contract drawings. Their arrangement is such that the detailer can trace them directly on the final plans.

*Report of ACI Committee 317, 1939.

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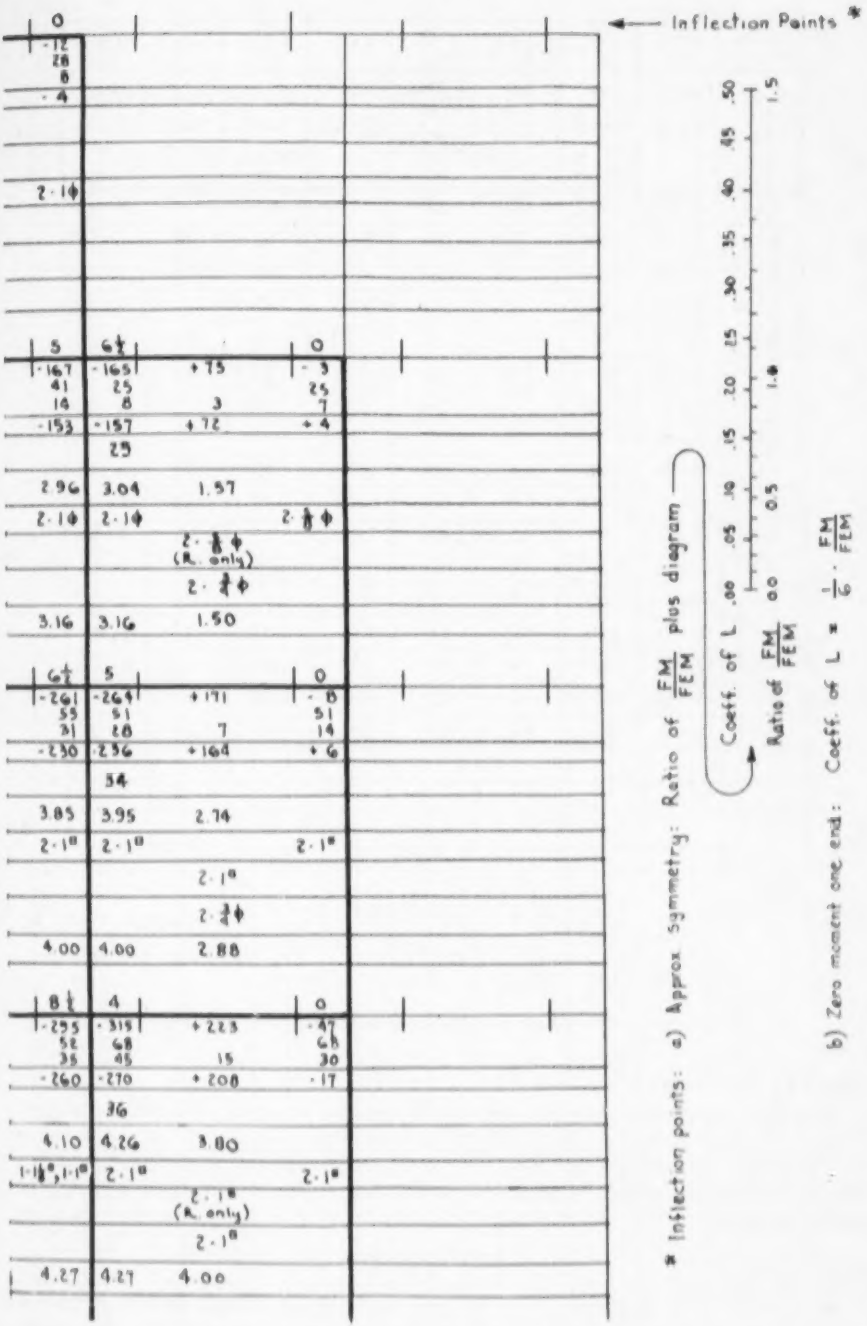


Sheet 7—(continued from opposite page)—**Design of beams**

①	②	③	④	⑤	⑥	⑦	⑧	⑨
Col.	Est. Dim b × e	N Axial	M	$\frac{CD^{*}M}{e}$ $\frac{CD^{*}④12}{②}$	P ⑤+③	Load on Conc. Table 18	Load on Steel ⑥-⑦	Steel Table 18
A 32	12 × 10	33	4	14	47	65	-	4-$\frac{5}{8}\phi$
A 21	14 × 10	105	6	22	127	76	51	6-$\frac{3}{4}\phi$
A 10	16 × 12	175	20	60	235	104	131	8-1ϕ
B 32	12 × 10	32	2	7	39	65	-	4-$\frac{5}{8}\phi$
B 21	14 × 10	88	3	11	99	76	23	4-$\frac{5}{8}\phi$
B 10	16 × 10	141	8	30	171	86	85	6-1ϕ
C 43	12 × 10	38	12	36	74	65	9	4-$\frac{5}{8}\phi$
C 32	12 × 10	103	3	11	114	65	49	4-$\frac{3}{4}\phi$
C 21	14 × 14	168	6	15	183	106	77	4-1$\square$
C 10	16 × 16	230	14	31	261	138	123	4-1$\frac{1}{4}\square$
D 43	12 × 10	38	12	43	81	65	16	4-$\frac{5}{8}\phi$
D 32	12 × 12	126	2	6	132	78	54	4-1ϕ
D 21	14 × 20	235	9	16	251	151	101	4-1$\frac{1}{8}\square$
D 10	16 × 24	359	18	27	386	207	179	6-1$\frac{1}{4}\square$
E 32	12 × 10	35	4	14	49	65	-	4-$\frac{5}{8}\phi$
E 21	14 × 10	114	6	22	136	76	60	4-1ϕ
E 10	16 × 16	210	41	92	302	138	164	6-1$\frac{1}{4}\square$

* CD = 3 for Tied Columns
4 for Square Spiral Column
6 for Round Spiral Column
} Assumed tied this problem

Sheet 8—Design of columns

Sheets 10 and 11 cont'd

Note that a minimum distance of two feet is used for all inflection points. Also note that in the short seven-foot spans, inflection points are shown to exist at a distance of two feet from the column centerline. Of course, these beams have no inflection points but this arbitrary distance is used to establish cut-off points for negative steel from adjacent beams.

4. CONCLUSIONS

The total time necessary to fill out the eleven blank forms required about twenty hours, or two and one-half working days. To this must be added the time necessary to check the work and make corrections.

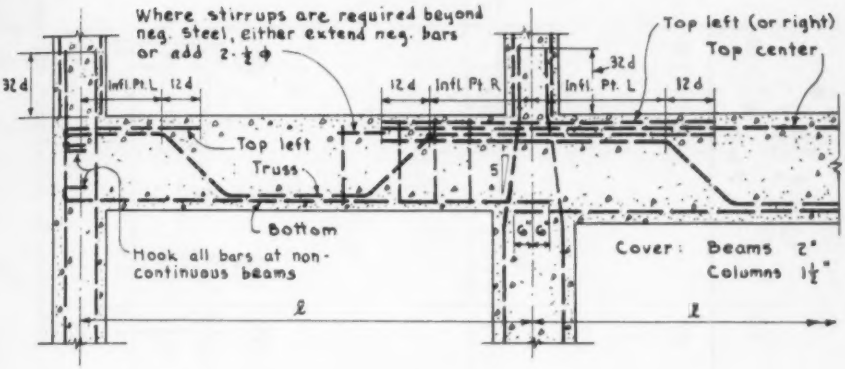
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫	⑬	⑭	⑮
Beam	b	d	ℓ	V_{sim}	V_{cor}*	V ⑤ + ⑥	v ⑦ / (② × ⅞ × ③)	v' ⑧ − .02f'_c	S (⑨ / ⑧) × (④ / 2)	Stirrup Size	$A_s f_y$ Diagram 17	Max $\frac{1}{5}$ ⑨ × ② / ⑫	Index 1.5 × ⑩ / ⑬	Stirrups Required Diagram 17
4CD	12	29	25	28	0	28	92	32	4.4	⅜ ϕ	5280	0.013	130	1/6 4/12
3AB	12	29	20	23	4	27	89	29	3.3	⅜ ϕ	5280	0.066	75	1/6 3/12
3BC	12	29	7	7	20	27	88	28	1.1	⅜ ϕ	5280	0.066	25	1/6 1/12
3CD	12	29	25	41	2	43	142	82	7.2	⅜ ϕ	5280	0.186	58	1/3 3/6 3/8 3/12
3DE	12	29	22	25	7	32	105	45	4.7	⅜ ϕ	5280	0.102	69	1/6 4/12
2AB	14	34	20	44	7	51	122	62	5.1	⅜ ϕ	5280	0.165	46	1/3 1/6 2/8 3/12
2BC	14	34	7	12	12	24	58	-	-	-	-	-	-	-
2CD	14	34	25	55	5	60	144	84	7.3	⅜ ϕ	5280	0.233	49	1/2 2/4 4/6 2/8 3/12
2DE	14	34	22	51	12	63	151	91	6.6	½ ϕ	9600	0.132	75	1/4 2/8 5/12
1AB	16	24	20	42	6	48	143	83	5.8	⅜ ϕ	5280	0.252	35	1/2 4/3 2/6 2/8 2/12
1BC	16	24	7	11	4	15	45	-	-	-	-	-	-	-
1CD	16	36	25	52	12	64	127	67	6.6	⅜ ϕ	5280	0.203	49	1/3 3/6 2/8 3/12
1DE	16	36	22	68	12	80	159	99	6.9	½ ϕ	9600	0.164	63	1/3 1/6 3/8 4/12

* Correct for end beams. Values doubled for interior beams. [End moments taken from sheet 5]

Sheet 9—Design of stirrups

Beam	ℓ	w	d	Truss	Bottom	Top			Infl. Points		Stirrups		Remarks
						Left	Center	Right	Left	Right	Size	Spacing	
4CD	25	12	29	2-1φ	2-1φ				2'	2'	$\frac{3}{8}$φ	1/6 4/12	
3AB	20	12	29	2-$\frac{5}{8}$φ	2-$\frac{5}{8}$φ				2'	3'	$\frac{3}{8}$φ	1/6 3/12	Truss left side only
3BC	7	12	29		2-$\frac{5}{8}$φ		2-1φ		2'	2'	$\frac{3}{8}$φ	1/6 1/12	
3CD	25	12	29	2-1φ	2-$\frac{5}{8}$φ				4½'	5'	$\frac{3}{8}$φ	1/3 3/6 3/8 3/12	
3DE	22	12	29	2-$\frac{5}{8}$φ		2-1φ			6½'	2'	$\frac{3}{8}$φ	1/6 4/12	Truss right side only
2AB	20	14	34	2-$\frac{3}{4}$φ	2-$\frac{7}{8}$φ				2'	3½'	$\frac{3}{8}$φ	1/3 1/6 2/8 3/12	
2BC	7	14	34		2-$\frac{5}{8}$φ		2-1φ		2'	2'	-		
2CD	25	14	34	2-1□	2-$\frac{5}{8}$φ				4½'	6½'	$\frac{3}{8}$φ	1/2 2/4 4/6 2/8 3/12	
2DE	22	14	34	2-1□	2-$\frac{3}{4}$φ				5'	2'	$\frac{1}{2}$φ	1/4 2/8 5/12	
1AB	20	16	24	2-1φ	2-1φ			2-$\frac{5}{8}$φ	2'	3½'	$\frac{3}{8}$φ	1/2 4/3 2/6 2/8 2/12	Truss left side only
1BC	7	16	24		2-$\frac{5}{8}$φ		2-1$\frac{1}{8}$□		2'	2'	-		
1CD	25	16	36	2-1φ	2-$\frac{5}{8}$φ			1-1$\frac{1}{8}$□ 1-1□	3½'	8½'	$\frac{3}{8}$φ	1/3 3/6 2/8 3/12	Truss left side only
1DE	22	16	36	2-1□	2-1□	2-1□			4'	2'	$\frac{1}{2}$φ	1/3 1/6 3/8 4/12	Truss right side only

Sheet 10—Beam schedule



4						
3			12 x 10 4-⅝ φ	12 x 10 4-⅝ φ		
2	12 x 10 4-⅝ φ	12 x 10 4-⅝ φ	12 x 14 4-⅞ φ	12 x 12 4-1 φ	12 x 10 4-⅝ φ	
1	14 x 10 6-¾ φ	14 x 10 4-⅝ φ	14 x 14 4-1 □	14 x 20 4-1⅛ □	14 x 10 4-1 φ	
0	16 x 12 8-1 φ	16 x 10 6-1 φ	16 x 16 4-1¼ □	16 x 24 6-1¼ □	16 x 16 6-1¼ □	
	A	B	C	D	E	

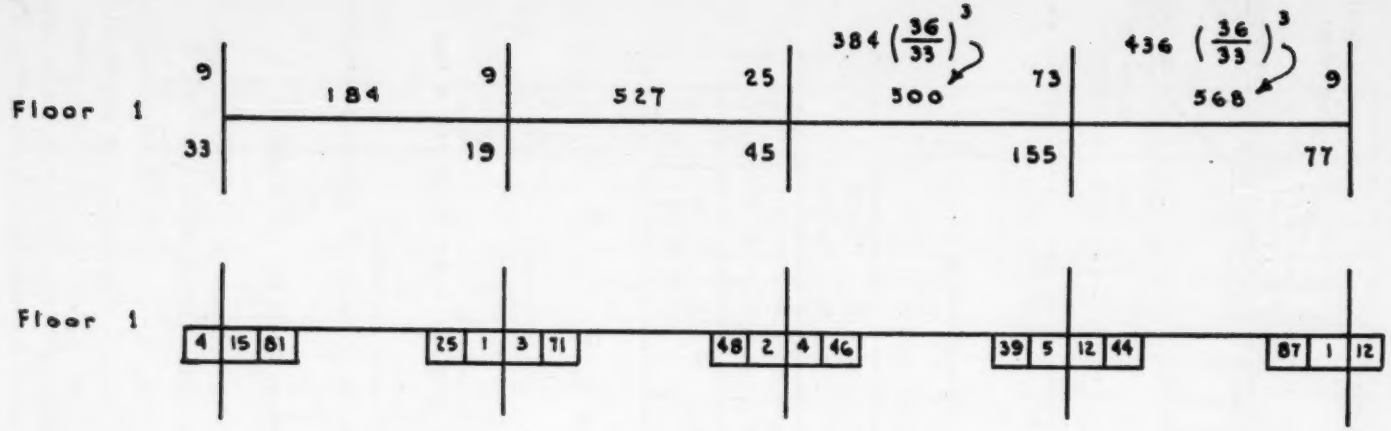
Column Notes

1. Lap bars 32 d above floor
2. All columns to have ⅜ φ ties @ 12" oc. Use 1 tie for 4 bars, 2 for 6, 3 for 8, etc.
3. First dimension is normal to plane of frame.
4. Arrange half of steel along each face of dimension normal to frame.

Sheet 11—Typical beam and column details and column schedule

Since thirteen beams and seventeen columns have been completely designed, the time required does not seem unreasonable.

Some concern may be felt regarding the necessity of completely redesigning a frame because the calculated dimensions are appreciably different from the preliminary sizing. Reference to Check Sheet 1 should minimize concern on this point, provided the predesign is based on *FEMs*. Admittedly, there is room for considerable study on this



DF bm	(81)			(25)		(71)			(48)	(46)			(39)	(44)			(87)		
FEM D	-77			-77		-5			-5	-120			-120	-97			-97		
FEM T	-140	+70		-140		-13	+7		-13	-218	+109		-218	-250	+125		-250		
	-17	→		-57		+26	→		+23	-24	→	+18	↙ -49	-109	→	+79	↙ -29		
	-157		↙	-197		+13		↙	+10	-242		+34	↙ -267	-359		+25	↙ -279		
	+127			+53		-149			-121	+116			-36	+40			+242		
Final Mom	-30			-144		-136			-111	-126	+161		-303	-319	+225		-37		
From Sheet 5	-30			-144		-135			-125	-143	+153		-295	-315	+223		-47		

Check Sheet 1—K-values, distribution factors at joints, maximum positive and negative moments—Moment comparison for revised beam stiffnesses

feature of the procedure, particularly where variable beam depths are to be used.

A casual inspection of the work reveals that few new ideas or thoughts have been introduced in this procedure. It is essentially a careful arrangement of existing methods. As a result, it can be readily understood and applied by inexperienced engineers.

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Discussion of a paper by D. R. Cervin:

A Practical Procedure for Rigid Frame Design*

By W. D. BIGLER, PHIL M. FERGUSON and AUTHOR

By W. D. BIGLER†

Mr. Cervin's procedure for designing concrete buildings by tabulating computations and dimensions on prepared forms is compact and efficient. Every structural engineering office needs such a system to reduce work and errors to a minimum.

The two-cycle approximate method of moment distribution that the author recommends is obviously a great time saver, sufficiently accurate for the usual type of rectangular building frame. This method assumes that ends of beams are fixed two spans from the joint under consideration and that the far ends of the columns are fixed. It consequently has the limitation that unbalanced beam moments transmitted to columns do not enter into computation of beams above and below. Moreover, the assumption that a column is necessarily fixed at a footing may lead to error. Especially if the column rests on a single-pile footing or on a relatively narrow continuous foundation, the lower end actually is only slightly restrained, if not hinged.

As a second limitation, the two-cycle method does not consider the effect of lateral deflection or "sidesway" resulting from an adjustment of the frame so that internal shears equal zero. It is true, however, that in wide, multistoried buildings and in small buildings not considerably unsymmetrical in shape or loading, stresses resulting from lateral deflection are negligible.

To study the accuracy of the two-cycle method under unusual, but not improbable, conditions the writer selected a portion of the frame used by Mr. Cervin as shown in Fig. A. Dimensions, K values, and fixed-end moments are the same as those in the original example. First-

*ACI JOURNAL, Apr. 1945; *Proceedings* V. 41, p. 453.
†Long Beach, Calif.

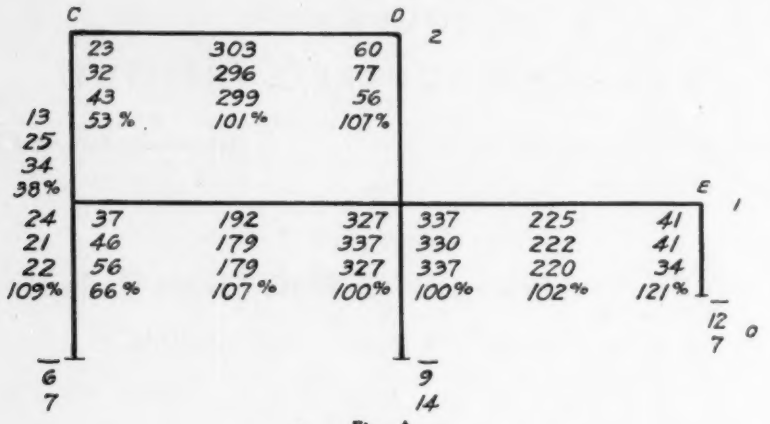

Fig. A

story columns are assumed half-fixed at footings. In this selection the K of wall column D12 is three times the K of wall column C12, but the dimensions of a square section need to be increased only 32 percent to triple the I, and such increase might result from wall crane loads. K values equally unbalanced as those in this example would exist in a building frame on the side of a hill with all columns of unequal length but with normally proportioned sections.

Fig. A shows a comparison of positive and negative beam and exterior column moments from three sets of computations. Starting with the upper, the four tabulated figures are:

1.–Maximum moment computed by the two-cycle method.

2.–Maximum moment selected from computations by the Cross method on all seven possible patterns of live loading. This value, like the figure above, is based on the assumption that there exist forces at C1 and C2 which prevent lateral deflection.

3.–Maximum moment (No. 2 above) corrected for lateral deflection. This is the true moment.

4.–The percentage of the value obtained by the two-cycle method as compared with the true value.

These figures show only a negligible discrepancy in negative interior and positive beam moments. However, in exterior negative beam and exterior column moments the approximate method shows a maximum error of −62 per cent. It is apparent, therefore, that small frames considerably unsymmetrical in shape or loading require exact analysis. As the author suggests, such procedure is laborious. The designer can keep the work at a minimum by sketching the elastic line of the continuous frame as it deflects when one beam or two adjoining beams are loaded and thus find the patterns of loading that produce maximum moments.

Often more convenient for solving small frames is the method described in the following six steps:

1.–Distribute the total fixed-end moments of one beam, considering no load, either live or dead, to be on the other beams. To lessen the labor involved, neglect all distributed moments farther away than, horizontally, two spans or one column length plus one span and, vertically, one column length from the beam under consideration. These discarded moments will be negligible. Three cycles of distribution are sufficient. Three are necessary, however, to include returns from the column ends.

2.–Correct for unbalanced shear by one of the methods described below.

3.–Compute the positive moments at midspan, which equal $FEM_T \times 1.5 - \frac{FM_L + FM_R}{2}$ for uniform loading.

4.–Repeat the procedure for all other beams, one at a time.

5.–Tabulate on a diagram of the frame, at its proper location, the various partial moments thus obtained and beside each the product of the partial moment times $\frac{FEM_D}{FEM_T}$. The second figure is the corresponding partial moment for dead load alone.

6.–At each tabulation, select the combination, one from either the left or right figure of each pair, that will add to the maximum. The sum is the true maximum moment.

The maximum stress in a column is $\frac{P}{A}$ plus the maximum compressive stress due to bending. In computing P, consider live load on all those beams framing into the column which are not represented in the tabulation by dead-load moments alone.

For correcting unbalanced shear, step No. 2 above, a combination of the Morris method* with the Cross method will keep the shear at approximately zero during distribution of *FEMs*. If more than two-place accuracy is used, however, it is more convenient to carry out the three-cycle distribution of *FEMs* first and then find the shear corrections. A third method, more easily understood and less liable to error, and usually less laborious, especially in a lower story that has columns of unequal length, is as follows:

Find $TM = -S \times F \times \frac{R}{\Sigma R} \times \frac{h}{2}$ for all columns, in which TM = trial cor-

*See discussion by Prof. C. T. Morris, Trans. A. S. C. E., vol. 96, page 68.

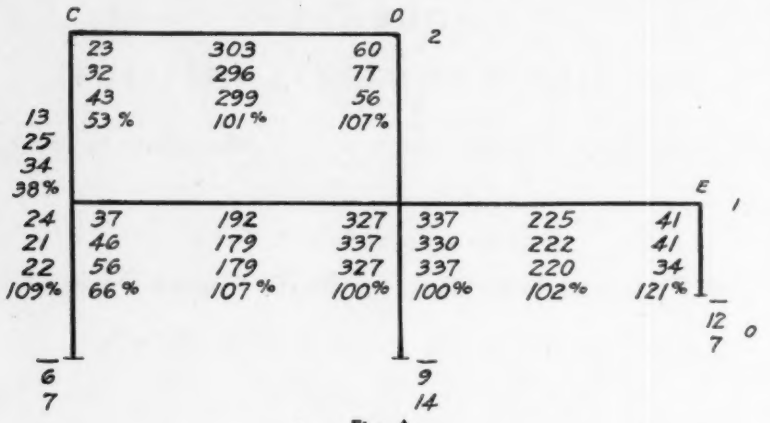

Fig. A

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Find $TM = -S \times F \times \frac{R}{\Sigma R} \times \frac{h}{2}$ for all columns, in which TM = trial cor-

*See discussion by Prof. C. T. Morris, Trans. A. S. C. E., vol. 96, page 68.

recting moment to be distributed from a column end. The two values for any one column are equal.

$S = \sum \frac{FMca + FMcb}{h}$, the unbalanced shear in a story.

FMca and *FMcb* = moments at upper and lower end of column after distribution of *FEMs*.

$R = \frac{I}{h^3}$ of a column.

$\frac{R}{\Sigma R}$ = proportional part of shear in a story taken by a column.

$F = \frac{1}{2N} \times \sum \frac{1}{1\text{-}nD}$ for the story under consideration. There are two values of $\frac{1}{1\text{-}nD}$ for each column. One value of *F*, greater than unity, applies throughout a story and each story will have a different value.

n = number of columns at a joint.
N = number of columns in the story.
D = distribution factor of column.
If all columns in the story have the same length, then:

$TM = \frac{-Mc \times F}{2} \times \frac{R}{\Sigma R}$, in which

Mc = algebraic sum of the moments at the upper and lower ends of the colums after distribution of *FEMs*.

Distribute *TMs* and add the corrections thus derived to the moments already obtained. If further corrections are necessary, repeat the procedure with *S* equal to the remaining unbalanced shear. No repeated correction was necessary in solving for side way the frame shown in Fig. A.

The author has not made provisions in his forms for checking frames for wind loads. One or two sheets can be added conveniently for comparing bending moments and direct stresses due to wind and vertical loads.

By PHIL M. FERGUSON*

The author's re-emphasis on the very great time saving possible by use of the two-cycle moment distribution method (originally developed by the Portland Cement Association) is a definite contribution toward

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more rational designs. This has been coupled with tabular forms which greatly systematize the auxiliary calculations.

Moments by the two-cycle method will not be "exact" when applied to irregular spans and loadings. The two-cycle method does not consider any rotation of joints at the floors above or below the one under investigation, nor does it consider the rotation of more than three joints in the calculation of any one maximum moment. To illustrate the results of this procedure, where spans are irregular as in this case, the analysis was repeated for maximum loadings, this time using the entire second floor and multiple distributions at all five joints. These maximum moments are tabulated and compared below.

Method	A	B	C	D	E
Entire floor....	−8+145*−157	−156−54−175	−185+148−269	−270+164*−9	
Two cycle.....	−7+135 −152	−150−53−191	−198+146−261	−264+171 −8	
		Min. positive			
Entire floor....		−140			
Two cycle.....		−116			

*At mid-span. Maximum is off center, amounting to +150 for AB and + 169 for DE

None of these errors is serious, although the variation at C is noticeable and the minimum positive moment in BC by the two-cycle method is rather on the unsafe side. Lest the error in moments at C be thought to typify general errors that will *always* be on the side of safety, the following values of maximum moment at C on the third floor are tabulated:

	C	
Entire floor.....	−156	−162
Two cycle.....	−146	−150

Here the error is on the unsafe side. With major differences in spans and in loadings adjoining C, the results of the two-cycle method must be considered good. It is to be hoped that more engineers will utilize this method to improve their design data.

Since the author proposes his method as a model that others may follow, one error of serious import should be noted. Sheet 7 shows several beams (especially beams CD at different levels) where approximately three-fourths of the positive moment steel is provided in the form of trussed bars. In my experience, no case of beams this length is recalled where it would have been possible to truss or bend up three-quarters of the positive steel without infringement on either the positive or negative moment needs. Since these are deep beams, it seems unlikely that much over half the area of the positive steel can be bent up

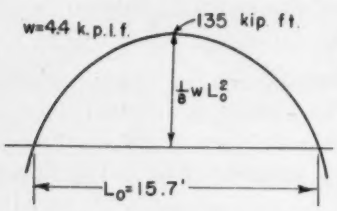

Fig. B (left) Max. Positive Moment.

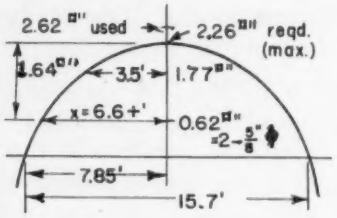

Fig. C (right) Reqd. A_s Curve

and made available for negative steel. The 1940 Joint Committee Specification will not often permit the trussing of as much as half the bottom steel (when beams are designed for uniform loads). The bend down points specified on Sheet 11 (i.e., at the point of inflection which accompanies maximum negative moment loading) are entirely unsatisfactory for usage in ordinary building construction, unless it be for shallow construction such as slabs.

The unsatisfactory condition in the extreme case of beam CD at the second floor level will be shown numerically. The maximum positive moment of 135 kip ft. (Sheet 7) exists very near the center line of the span. Since w_t is 4.4 kips p.l.f., the distance *Lo* between points of inflection (Fig. B) can be obtained from the relation $135 = \frac{1}{8} \times 4.4 \times L_o^2$, or $L_o = \sqrt{\frac{1080}{4.4}} = 15.7$ ft. The moment curve can be used as the required A_s curve, as shown in Fig. C. The *required* steel, 2.26 sq. in. (Sheet 7) has been used as the maximum ordinate, instead of the total area of 2.62 sq. in. which was provided, since this permits slightly shorter lengths for that part of the bottom steel that is bent up. This curve shows that some part of the 2-1 in. sq. bars must be continued out a distance $x = 7.85\sqrt{\frac{1.64}{2.26}} = 6.6^+$ ft. If the 1-in. sq. bars are bent down at 45 deg. according to Sheet 11, at 6.5 ft. from the center of column D, they will be available at the level of the positive steel at 6.5 + 2.5 (assuming 30-in. offset) = 9.0 ft. from the center of column or 3.5 ft. from the middle of span. This compares with the needed 6.6 ft. and leaves a length of 6.6 − 3.5 = 3.1 ft. in the bottom with deficient steel. Stated another way, the steel needed at 3.5 ft. from the middle of the span is $2.26\left[1 - \left(\frac{3.5}{7.8}\right)^2\right] = 2.26 \times 0.788 = 1.77$ sq. in. Since the author provides at this point only 0.62 sq. in. or 35 per cent of the required area, a steel stress well beyond the yield point is indicated on the straight bottom bars.

The situation could be helped, but not completely remedied, by bending these particular bars down at *about* 3.5 ft. from the center of the column.

The typical bend point called for on Sheet 11 should be moved closer to the column, and the steel on Sheet 7 should be selected with the extent of the positive moment requirements more in mind.

Two other comments will be made, neither so serious as the above. The author's method of calculating V_{con} on Sheet 9 by using *double* the difference between the *maximum* end moments, is new to me. It seems to work well for the frame shown. However, it should be noted that such a shear does not reduce to zero in the case of beams of uniform length and with equal live load, even though the maximum negative moments at the two ends of such a beam would be equal. Possibly some arbitrary *minimum* V_{con} should be taken into account, say 10 per cent of the simple span shear as a minimum for the case when the maximum end moments are nearly the same.

The value of V_{con} calculated by the author's method will add to the simple beam shear only on that half of the beam adjacent to the *larger* negative moment. On the half of the beam adjacent to the smaller negative moment this particular value of V_{con} will subtract from the simple beam shear, but this reduced shear would not be the maximum. A different loading of adjacent spans, the same as that for maximum negative moment at this end, is required for the maximum shear, and this loading gives a different value of V_{con}. This value of V_{con} will be smaller than the author's value and may even be of opposite sign to the simple beam shear at this end, thus sometimes giving a design shear adjacent to the smaller negative moment that is less than the simple beam shear. Since V_{con} is not large, except when the difference in the maximum negative moments at the ends of the beam is large, the differences discussed here will not usually be important, and some designers will prefer to ignore them. However, in beam DE on the second floor, more complete calculations show that the maximum total shear at the E end is really $-51 + 10 = -41$ kips, instead of the 63 kips shown on Sheet 9.

The author selects his beam sizes by determining the balanced rectangular beam size necessary to care for the maximum negative moment. Since some bottom steel always runs into the support, it would seem economical to reduce beam depths (and thereby structure height) by utilizing this straight bottom steel as compressive reinforcement at the face of columns.

In summary, the two-cycle moment distribution method is a distinct advance in design procedure and is entirely practical. However, there is an element of danger always present when the detailer and designer

of continuous structures are too far apart. A good designer must know his details, and a good detailer can scarcely proceed by rule of thumb alone. Poor detailing can be and often is as great a source of error as approximate moment assumptions.

AUTHOR'S CLOSURE

The writer acknowledges the two complete discussions of his paper "A Practical Procedure for Rigid Frame Design," submitted by Mr. W. D. Bigler and Prof. Phil M. Ferguson. Both papers are concrete in their discussion and have some genuinely helpful criticisms or suggestions to offer. Of considerable interest to the writer is the frank and candid opinion of both men in accepting and advocating the use of two-cycle moment distribution for continuous frames. That exceptions to its use can and will occur has been carefully explained by both commentaries.

Professor Ferguson has properly detected what he calls a serious error regarding the amount of positive steel and the bend down point of the negative steel. His own suggestion that the steel should be selected with the extent of the positive moment requirements more in mind, coupled with his very clear computations of how much positive steel should exist at various parts of the beam, provides a clue for systematically checking this feature as a part of the beam design on Sheet 7.

Between the lines "Straight, bottom" and "Steel provided" should appear this equation:

$$L_o = \sqrt{\frac{8\ (+\ M_{max})}{w_t}}$$

where L_o is defined in Symbols and Notation as the length of the beam having positive moment.

Applied to the Beam 3CD in question, this length is 15.7 feet. Half of this distance, measured from the centerline of the beam is therefore the point of inflection for positive steel. By bending up one bar at this point, it will develop useful negative steel at a distance of 2.0 feet from the centerline of the column. Since the parabolic shape of the negative moment is practically triangular, the first turn-down for negative moment can be made at one quarter of the negative inflection point, or 1.3 feet, somewhat less than the point already selected. It is then necessary to determine if sufficient steel exists after the first trussed bar to satisfy the positive moment requirements. For the beam in question Prof. Ferguson has already computed that 1.77 sq. in. are necessary, and the steel provided by one 1-in. square and two-⅝-in. round is 1.62 sq. in., less than ten per cent deficient. If it now be tacitly assumed that Prof. Ferguson's first suggestion regarding care in selection of positive steel be

adopted, no such variation in positive and negative steel would exist as demonstrated in this example. Thus, the ten per cent deficiency estimated above would conceivably be a lesser value, and any happier choosing of positive and negative steel areas would minimize or completely remove this deficiency. It hardly seems necessary to perform any calculations in routine problems other than substituting in the formula already proposed; doubtful cases should be checked analytically. Care should be taken, however, to have the bend down point of the steel nearest the column centerline not closer than one fourth of negative inflection point. It is reasonably apparent that turning up all the trussed bars slightly before reaching the positive inflection point is not serious since at least one third of positive steel area should be carried through to the supports. Only a small change is necessary on the beam schedule and on the typical beam drawing to accommodate this addition.

Regarding the calculation of V_{con}, Professor Ferguson's suggestion that some arbitrary *minimum* V_{con} of about 10 per cent of V for the simple span be used, is based on sound reasoning and is worthy of adoption. However the computation of the lesser shear of the two ends of a beam seems hardly worthwhile. It is doubtful if the average beam would save over a single stirrup by such a procedure; and the inconvenience of dissymmetrical detailing coupled with the additional computations involved, would hardly balance the steel saved.

Prof. Ferguson's last suggestion proposed that the compressive reinforcement at the face of the column be considered in computing beam depths as a means of economizing in concrete and overall building height. Again the suggestion is in order, but care must be taken to devise a tabular method of solution which permits rapid design of many beams. Any method used depends upon the individual tastes of the designer; the writer believes that the method recommended in "Reinforced Concrete Design Handbook"* lends itself well to mass use on Sheet 7.

Mr. Bigler has as the kernel of his discussion, the effect of lateral deflection or "sidesway" resulting from an adjustment of the frame so that internal shears equal zero. The comparative results of a highly dissymmetrical frame analyzed by two-cycle moment distribution, Cross moment distribution without correction for sidesway, and Cross distribution corrected for sidesway (See Fig. 1), is truly revealing in that some types of frames or loadings warrant an exact analysis.

However, the results of Mr. Bigler's computations do clearly reveal that the only major errors are in the outside negative moments of wall beams and in wall columns. Since the magnitude of end moments in wall columns is usually small, the degree of error here is of little consequence

*Reported by ACI Committee 317.

because the available steel from other moment considerations is usually far more than required. The same favorable considerations are not applicable to wall columns. It can be pointed out, however, that the major influence on column design is axial load and not bending moment; therefore, moment deficiencies of 50 per cent obtained by two cycle distribution will result in an overall column design deficient only about 10 per cent.

This is not to be construed as a brief defending slipshod or faulty methods of design in order to save engineering time. It must be remembered that the purpose of this paper was to layout a practical procedure for rigid frame design. Most designers are acquainted with some method of sidesway correction, but it is very doubtful if many are prepared to spend the time required for such work. Mr. Bigler shows considerable versatility in handling lateral deflection, but even he admits a large amount of work to achieve his results.

Until a method of correcting for sidesway is developed which has the comparable speed, simplicity, and thoroughness of two-cycle moment distribution, it does not appear feasible to this writer to make the desired correction for lateral deflection. The challenge is left to Mr. Bigler and other capable theoreticians to solve this problem so that all practicing engineers can obtain fully accurate results consistent with a reasonable time expenditure.

[To facilitate selective distribution, **separate prints** of this title (41-20) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than July 1, 1945.]

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Dynamic Testing of Pavements*

By GERALD PICKETT†

SYNOPSIS

A theoretical analysis is made of the problem of the vibration of a pavement in contact with an elastic subgrade. The analysis shows that for each frequency of sustained vibration, waves may be propagated horizontally with three different velocities. The properties of the subgrade have very little effect on the highest or the lowest of these three velocities but have considerable effect on the intermediate velocity. Near the origin the velocity of each wave may be greater or less than the velocity farther from the origin. The analysis indicates that the properties of both pavement and subgrade may be determined for any small region of the pavement by placing the source of sustained vibrations in that region.

INTRODUCTION

The dynamic method of determining Young's modulus of elasticity of concrete has been used for several years as a laboratory test.[1-4] A specimen under test is caused to vibrate at one or more of its resonant frequencies while being supported either on rubber or in such a way that the support has a negligible effect upon the results. Equations giving the approximate value of these resonant frequencies in terms of the weight, elastic constants, and dimensions of the specimen are available for the usual shapes of laboratory specimens, and thus the modulus of elasticity can be readily computed after the resonant frequency has been determined experimentally.

The application of dynamic testing to concrete pavements and structures in place has probably been retarded both by lack of suitable measuring instruments and by the lack of the necessary mathematical expressions for interpreting the results of measurements. Bernhard[5] compared the vibration properties of pavement slabs of different thicknesses. He used a mechanical oscillator with a frequency range of from ½ to

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[1-4] See references at end of paper.

40 cycles per second. Recently, Long, Kurtz, and Sandenaw[6,7] have developed instruments that give reasonable accuracy for the time of travel of waves between two points on the surface of a pavement slab, even when the points are no more than a foot apart. They measured velocity of waves produced by impact and also those produced by sustained vibration at frequencies ranging from 1000 to 2000 cycles per second. They also made some progress in interpreting the results in terms of Young's modulus of elasticity of the pavements by the use of equations developed by Lamb[8] and Timoshenko.[9]

The accomplishments noted above and the fact that the use of the equations of Lamb and Timoshenko for this purpose may be questionable stimulated the author to prepare the present paper.

LIMITATIONS OF PREVIOUS EQUATIONS

The equations of Lamb and Timoshenko are not strictly applicable to pavements because:

(1) They are for freely vibrating slabs, whereas the pavement is in contact with the subgrade.

(2) They are for a two-dimensional problem (plane waves), whereas the vibration of a pavement is a three-dimensional problem.

In general both the shape and the support of field structures are such as to make analysis of vibration more complicated than that for small laboratory specimens.

So far as is known, the nearest analytical approach to the vibration of a pavement was a study made by Love[10] in which he investigated the vibration of the earth's crust, assuming that the elastic properties of the crust differ from those of the interior. However, Love's studies were also confined to two-dimensional problems and only dealt with cases in which the crust was less rigid than the interior, whereas a concrete pavement will usually be more rigid than its subgrade.

SCOPE OF PRESENT DISCUSSION

This paper will discuss certain possible modes of vibration of a pavement in contact with its subgrade that are likely to occur in the dynamic testing of pavements. An equation giving the relation between driving frequency, thickness of slab, elastic constants of both pavement and subgrade, and the velocity of wave is derived on the assumption of continuity of motion between pavement and subgrade. The equations of elasticity applicable to a homogeneous, isotropic, elastic solid are used for both the slab and the subgrade.

If experiments prove that the equations are generally applicable to pavements in place, then from a few dynamic measurements it should

be possible to determine not only Young's modulus for the concrete but also the thickness of the pavement and a modulus for the subgrade.

PARTICULAR SOLUTIONS OF THE DIFFERENTIAL EQUATIONS OF VIBRATION

When expressed in the cylindrical coordinates r, Θ, z the differential equations of vibration for a homogeneous, isotropic, elastic solid are:[11]

$$(\lambda + 2G)\frac{\partial \Delta}{\partial r} - \frac{2G}{r}\frac{\partial W}{\partial \Theta} + 2G\frac{\partial V}{\partial z} = \rho\frac{\partial^2 u}{\partial t^2}$$

$$(\lambda + 2G)\frac{1}{r}\frac{\partial \Delta}{\partial \Theta} - 2G\frac{\partial U}{\partial z} + 2G\frac{\partial W}{\partial r} = \rho\frac{\partial^2 v}{\partial t^2}$$

$$(\lambda + 2G)\frac{\partial \Delta}{\partial z} - \frac{2G}{r}\frac{\partial}{\partial r}(rV) + \frac{2G}{r}\frac{\partial U}{\partial \Theta} = \rho\frac{\partial^2 w}{\partial t^2}$$

where u, v, w are displacements in the r, Θ, z directions, respectively;
t is time;

$$\Delta = \frac{\partial u}{\partial r} + \frac{u}{r} + \frac{1}{r}\frac{\partial v}{\partial \Theta} + \frac{\partial w}{\partial z};$$

$$2U = \frac{1}{r}\frac{\partial w}{\partial \Theta} - \frac{\partial v}{\partial z};$$

$$2V = \frac{\partial u}{\partial z} - \frac{\partial w}{\partial r};$$

$$2W = \frac{\partial v}{\partial r} + \frac{v}{r} - \frac{1}{r}\frac{\partial u}{\partial \Theta};$$

ρ = mass density;

$\lambda = \dfrac{\mu E}{(1 + \mu)(1 - 2\mu)}$, Lamé's constant;

$G = \dfrac{E}{2(1 + \mu)}$, modulus of elasticity in shear;

E = Young's modulus; and

μ = Poisson's ratio.

Four particular solutions of the differential equations will be used in the following discussions. Two of these, designated H and V, apply to the pavement slab, and two, designated H_1 and V_1 apply to the subgrade. The solutions will be used in pairs, the H solution for the pavement with the H_1 solution for the subgrade, etc. In the H and H_1 solutions points on the line $r = 0$ move in a horizontal direction and in the V and V_1 solutions these points move in a vertical direction. In the following all subscripts of unity except that designating a Bessel

function of the first order refer to the subgrade. (J_o and J_1 are Bessel functions.) The solutions are as follows:

$$\begin{aligned} u &= Z \sin\Theta \cos pt \left[J_o(ar) - \frac{J_1(ar)}{ar} \right] \\ v &= Z \cos\Theta \cos pt \frac{J_1(ar)}{ar} \\ w &= Z' \sin\Theta \cos pt\, J_1(ar) \end{aligned} \tag{H}$$

$$\begin{aligned} u_1 &= Z_1 \sin\Theta \cos pt \left[J_o(ar) - \frac{J_1(ar)}{ar} \right] \\ v_1 &= Z_1 \cos\Theta \cos pt \frac{J_1(ar)}{ar} \\ w_1 &= -Z_1' \sin\Theta \cos pt\, J_1(ar) \end{aligned} \tag{H$_1$}$$

$$\begin{aligned} u &= Z \cos pt\, J_1(ar) \\ v &= 0 \\ w &= -Z' \cos pt\, J_o(ar) \end{aligned} \tag{V}$$

$$\begin{aligned} u_1 &= Z_1 \cos pt\, J_1(ar) \\ v_1 &= 0 \\ w_1 &= Z_1' \cos pt\, J_o(ar) \end{aligned} \tag{V$_1$}$$

---

where $Z = M \cosh mz + M' \sinh mz + N \cosh nz + N' \sinh nz;$

$$Z' = \frac{Mm}{a} \sinh mz + \frac{M'm}{a} \cosh mz + \frac{Na}{n} \sinh nz + \frac{N'a}{n} \cosh nz;$$

$$Z_1 = Re\left[M_1 e^{-m_1 z} + N_1 e^{-n_1 z}\right];$$

$$Z_1' = Re\left[\frac{M_1 m_1}{a} e^{-m_1 z} + \frac{N_1 a}{n_1} e^{-n_1 z}\right];$$

$$m = a\sqrt{1 - a^2}, \qquad m_1 = a\sqrt{1 - a_1^2};$$

$$n = a\sqrt{1 - b^2}, \qquad n_1 = a\sqrt{1 - b_1^2};$$

$$a = \frac{p}{a}\sqrt{\frac{\rho}{\lambda + 2G}}, \qquad a_1 = \frac{p}{a}\sqrt{\frac{\rho_1}{\lambda_1 + 2G_1}};$$

$$b = \frac{p}{a}\sqrt{\frac{\rho}{G}}, \qquad b_1 = \frac{p}{a}\sqrt{\frac{\rho_1}{G_1}};$$

p = 2π times the frequency of vibration;

a = $p/V = 2\pi/l$ where V is the radial velocity which the wave approaches asymptotically as it gets farther from its source and l is the corresponding wave length; and

M, N, M', N', M_1, and N_1 are constants proportional to the amplitude of vibration.

Re [ ] signifies the real part of the expression in the bracket if either m_1 or n_1 is imaginary in a mathematical sense. If both m_1 and n_1 are real, then the Re in front of the brackets may be disregarded.

In each pair of solutions, for example, H and H_1, the frequency $p/2\pi$, the velocity p/a, the six amplitude constants M, N, M', N', M_1, and N_1, and the physical properties are arbitrary. That is, the differential equations are satisfied for any arbitrary values of these parameters. However, the boundary requirements at the top of the pavement and at the common boundary between pavement and subgrade permit the elimination of the six amplitude constants. The result is an equation, called the frequency equation, giving the relation between frequency, velocity of wave propagation and the physical properties of pavement and subgrade. It is of interest that the frequency equation is the same whichever pair of solutions is used and depends only on the assumptions made in regard to boundary conditions. In the derivation which follows the assumption is made that the top of the pavement is free of force and that the boundary stresses and displacements of the pavement are equal to those of the subgrade at their common boundary.

DERIVATION OF THE FREQUENCY EQUATION

The plane $z = 0$ is taken in the middle of a slab of thickness $2c$ and the direction $\Theta = 0$ is taken as due east as shown in Fig. 1.

The assumed boundary requirements result in the following relations:*

1. The top of the pavement is free of vertical force. $\sigma_z = 0$ at $z = -c$.
2. The top of the pavement is free of radial and tangential forces. $\tau_{rz} = \tau_{\Theta z} = 0$ at $z = -c$.
3. At the common boundary the vertical displacement of the pavement equals the vertical displacement of the subgrade. $w = w_1$ at $z = c$.
4. At the common boundary the radial and tangential displacements of the pavement equal the radial and tangential displacements, respectively, of the subgrade. $u = u_1$ and $v = v_1$ at $z = c$.
5. At the common boundary the vertical normal stresses are equal. $\sigma_z = \sigma_{1z}$ at $z = c$.

*Two relations are given in each of requirements 2, 4, and 6; but in each case the second relation is satisfied if the first is satisfied. Therefore the two relations give only one independent equation.

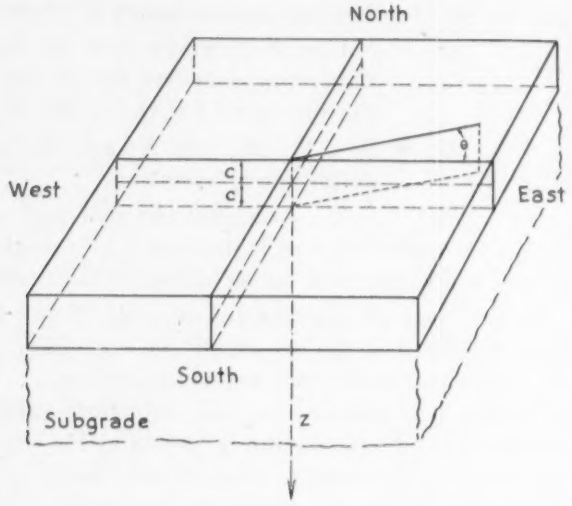

Fig. 1—Element of pavement slab and subgrade

6. At the common boundary the boundary shear stresses are equal. $\tau_{rz} = \tau_{1rz}$ and $\tau_{\Theta z} = \tau_{1\Theta z}$ at $z = c$.

When either pair of solutions for displacements is used and use is made of the relations previously given between m, n, λ, G, a, b, m_1, n_1, λ_1, G_1, a_1, and b_1 and of the following relations between stresses and displacements,

$$\sigma_z = \lambda \left(\frac{\partial w}{\partial z} + \frac{\partial u}{\partial r} + \frac{u}{r} + \frac{1}{r} \frac{\partial v}{\partial \Theta} \right) + 2G \frac{\partial w}{\partial z}$$

$$\tau_{rz} = G \left(\frac{\partial w}{\partial r} + \frac{\partial u}{\partial z} \right)$$

$$\tau_{\Theta z} = G \left(\frac{1}{r} \frac{\partial w}{\partial \Theta} + \frac{\partial v}{\partial z} \right) ,$$

the six boundary relations listed above result in the following six equations, respectively:

$$(2 - b^2)[M \cosh mc - M' \sinh mc] + 2[N \cosh nc - N' \sinh nc] = 0 \quad (1)$$

$$\frac{2m}{a}[- M \sinh mc + M' \cosh mc] + (2 - b^2) \frac{a}{n}[- N \sinh nc + N' \cosh nc] = 0 \quad (2)$$

$$\frac{m}{a}[M \sinh mc + M' \cosh mc] + \frac{a}{n}[N \sinh nc + N' \cosh nc] = - Re \left[\frac{M_1 m_1}{a} e^{-m_1 c} + \frac{N_1 a}{n_1} e^{-n_1 c} \right] \quad (3)$$

$$M \cosh mc + M' \sinh mc + N \cosh nc + N' \sinh nc = Re\,[\,M_1 e^{-m_1 c} + N_1 e^{-n_1 c}\,] \quad \ldots\ldots (4)$$

$$(2 - b^2)\,[M \cosh mc + M' \sinh mc] + 2[N \cosh nc + N' \sinh nc] = \frac{G_1}{G} Re\,[\,(2 - b_1^2)\,M_1 e^{-m_1 c} + 2N_1 e^{-n_1 c}\,] \quad \ldots\ldots (5)$$

$$\frac{2m}{a}[M \sinh mc + M' \cosh mc] + (2 - b^2)\frac{a}{n}[N \sinh nc + N' \cosh nc] = -\frac{G_1}{G} Re\left[2\frac{M_1 m_1}{a} e^{-m_1 c} + (2 - b_1^2)\frac{a}{n_1} N_1 e^{-n_1 c}\right] \quad \ldots\ldots (6)$$

The elimination of the six amplitude constants M, M', N, N', M_1, and N_1 from these six equations gives the following frequency equation:

$$\frac{h_1 f_1 (1 - A_1 - A_1')}{1 - A_2 - A_2'} - \frac{1 - A_3 - A_3'}{1 - A_4 - A_4'} = 0 \quad \ldots\ldots (A)$$

where $A_1 = \phi\left[\frac{2 - b^2}{2} \coth mc - k \coth nc + \frac{2 - b_1^2}{2}\frac{b^2}{2}\frac{\sqrt{1 - a^2}}{f_1}\right]$;

$$A_2 = \phi\left[\frac{2 - b_1^2}{2}\frac{2 - b^2}{2} \coth mc - \frac{2 - b_1^2}{2} k \coth nc + h_1 \frac{b^2}{2}\sqrt{1 - a^2}\right];$$

$$A_3 = \phi\left[\frac{2 - b_1^2}{2}\frac{2 - b^2}{2} \coth mc - \frac{2 - b_1^2}{2} k \coth nc + f_1 \frac{b^2}{2}\sqrt{1 - b^2} \coth mc \coth nc\right];$$

$$A_4 = \phi\left[\frac{2 - b^2}{2} \coth mc - k \coth nc + \frac{2 - b_1^2}{2}\frac{b^2}{2}\frac{\sqrt{1 - b^2}}{h_1} \coth mc \coth nc\right];$$

$$\phi = \frac{G_1}{2G\left[\left(\frac{2 - b^2}{2}\right)^2 \coth mc - k \coth nc\right]};$$

$k = \sqrt{(1 - a^2)(1 - b^2)}$;

$h_1 = \sqrt{1 - b_1^2}$ if $b_1^2 < 1$, i.e., if n_1 is real

$h_1 = -\sqrt{b_1^2 - 1} \cot (ac\sqrt{b_1^2 - 1})$ if $b_1^2 > 1$, i.e., if n_1 is imaginary

$f_1 = \sqrt{1 - a_1^2}$ if $a_1^2 < 1$, i.e., if m_1 is real;

$f_1 = \sqrt{a_1^2 - 1} \tan (ac\sqrt{a_1^2 - 1})$ if $a_1^2 > 1$, i.e., if m_1 is imaginary.

The expressions for A_1', A_2', A_3', and A_4' used in the frequency equation are the same as for the expressions for A_1, A_2, A_3, and A_4, respectively, except that in every instance coth is replaced by tanh.

THE MEANING OF IMAGINARY AND COMPLEX VALUES FOR m, n, m_1, AND n_1

The question of real and imaginary quantities entered into the solutions for the subgrade. The reason for this was that certain of the expressions became complex (contained both real and imaginary parts) under some conditions.

If the velocity of propagation of the waves horizontally is greater than the normal travel of disturbances within the subgrade, then either n_1 or both n_1 and m_1 become either imaginary or complex quantities in a mathematical sense. If internal friction is neglected as in the present study, then they become imaginary, but if internal friction is taken into account they become complex quantities. That is, if in the foregoing development p/a is greater than $\sqrt{G_1/\rho_1}$, then b_1 will be greater than unity and n_1 will be imaginary, and if p/a is also greater than $\sqrt{(\lambda_1 + 2G_1)/\rho_1}$, then m_1 will also be imaginary. When n_1 and m_1 are imaginary, the expressions $e^{-n_1 z}$ and $e^{-m_1 z}$ become trigonometric functions of the depth with both real and imaginary parts. For example, the real part of the expression for Z_1 becomes

$$Z_1 = M_1 \cos (az\sqrt{a_1^2 - 1}) + N_1 \cos (az\sqrt{b_1^2 - 1})$$

and for Z_1' becomes

$$Z_1' = M_1\sqrt{a_1^2 - 1} \sin (az\sqrt{a_1^2 - 1}) - N_1 \frac{\sin (az\sqrt{b_1^2 - 1})}{\sqrt{b_1^2 - 1}}$$

The factor n may also be imaginary, but such a possibility does not introduce any ambiguity into the solutions since cosh nz and (sinh nz)/n are both real whether n is real or imaginary. The factor m will ordinarily not be imaginary because the shear modulus of the pavement is greater than the shear modulus of the subgrade.

If either m_1 or n_1 is imaginary, then as just indicated, a part of the expression for a displacement within the subgrade will be a trigonometric function of the depth z. Consequently, this part of the expression will indicate no diminution of maximum amplitude of vibration with increase of depth in the subgrade. Had internal friction been taken into account, the solutions would indicate a decrease in amplitude with increase of depth. For example, if the assumption is made that internal friction is adequately considered by replacing the usual Hooke's law by equations of the type*

*The term with f has been added to the usual expression for Hooke's law such as given by Timoshenko in *Theory of Elasticity* (New York: McGraw-Hill, 1934), pp. 7-10.

$$e_x + f\frac{\partial e_x}{\partial t} = \frac{1}{E}\left[\sigma_x - \mu\sigma_y - \mu\sigma_z\right]$$

$$\gamma_{xy} + f\frac{\partial \gamma_{xy}}{\partial t} = \frac{\tau_{xy}}{G}$$

then solutions corresponding to those given in V_1 above for the subgrade can be written

$$u_1 = Z_1 \sin\Theta\, Re\left[e^{ipt}\left\{H_o^{(1)}(ar) - \frac{H_1^{(1)}(ar)}{ar}\right\}\right]$$

with similar expressions for v_1 and w_1, where the "real" Bessel functions of the first kind (J_o and J_1) have been replaced by "complex" Bessel functions of the third kind ($H_o^{(1)}$ and $H_1^{(1)}$)*; the "real" function cos pt has been replaced by the "complex" function e^{ipt}; and m_1 and n_1 have the values

$$m_1 = a\sqrt{1 - \frac{a_1^2}{1 + ifp}}, \qquad n_1 = a\sqrt{1 - \frac{b_1^2}{1 + ifp}}$$

in place of the values given previously.

A separation of m_1 and n_1 into their real and imaginary parts shows that each has a positive real part for all values of a_1 and b_1. Therefore, when internal friction of the subgrade is taken into account, the amplitude of vibration decreases with increase of depth.

In the above type of solution for vibration with internal friction the displacements and stresses are discontinuous at the line $r = 0$. It is on this line that the energy necessary to maintain sustained vibration is assumed to be supplied. To restrict the driving force to points on the pavement rather than at the line $r = 0$ would necessitate a still more complicated solution.

Since it is believed that internal friction has only a small effect on the velocity of propagation of the waves and since it was desired to keep the analysis relatively simple, friction was neglected in the derivation of the frequency equation.

NUMERICAL SOLUTION OF THE FREQUENCY EQUATION

The frequency equation derived above expresses the relation between the velocity† of wave propagation p/a, the frequency of vibration $p/2\pi$, and the physical properties of the pavement and subgrade. Unfortunately, this relation (Equation A) is rather involved, and numerical solution is not made readily. The method of solution found to be best in general was as follows:

*Bessel functions of first, second, and third kinds are treated in *Tables of Functions*, by Jahnke and Emde, 3d Ed. (New York: G. E. Stechert & Co., 1938), pp. 126-268.

†It must be remembered that the velocity of waves traveling radially depends on distance from the source. p/a is the velocity that they approach with increase of distance.

First, assume values for G_1/G, λ/G, λ_1/G_1, ρ_1/ρ.

Second, select a value of b, the ratio of velocity of propagation to $\sqrt{G/\rho}$.

Third, determine a, a_1, and b_1 and the functions of these quantities and of b that will be needed later such as $\sqrt{1-b^2}$, $\sqrt{1-b_1^2}$, $(2-b^2)/2$, etc.

Fourth, select a value of ac/π, the ratio of frequency times pavement thickness to the velocity of wave propagation.

Fifth, perform all indicated substitutions into the frequency equation, Equation A.

Sixth, select a second value of ac/π and make the indicated substitutions in Equation A.

Seventh, continue the process of selecting values of ac/π until Equation A is satisfied to the desired accuracy.

Eighth, select other values of b and repeat the above procedure as many times as seems desirable.

Results of such a procedure are shown in Fig. 2 where the velocity of wave propagation is plotted against frequency of vibration times thickness of subgrade.

Fig. 2 shows that there are three possible wave velocities for each value of frequency times thickness. The highest of these velocities is almost the same as the velocity of longitudinal waves in the pavement, if it were free of the subgrade. The lowest of these velocities is almost the same as the velocity of transverse (flexural) waves in the pavement if it were free of the subgrade. The intermediate velocity is between that of "Rayleigh waves" in the subgrade alone and that of Rayleigh waves in the pavement. At high frequencies all three velocities approach the velocity of Rayleigh waves in the pavement.

The fact that the highest and lowest velocities are practically independent of the properties of the subgrade is of considerable importance. For practical purposes it may therefore be unnecessary to obtain numerical solutions to the foregoing frequency equation. Instead, the following frequency equations derived by Lamb and Timoshenko will be adequate:

For the higher velocity (longitudinal vibration)

$$\left(\frac{2-b^2}{2}\right)^2 \coth mc = k \coth nc \quad \text{(L)}$$

and for the lower velocity (transverse vibration)

$$\left(\frac{2-b^2}{2}\right)^2 \tanh mc = k \tanh nc \quad \text{(T)}$$

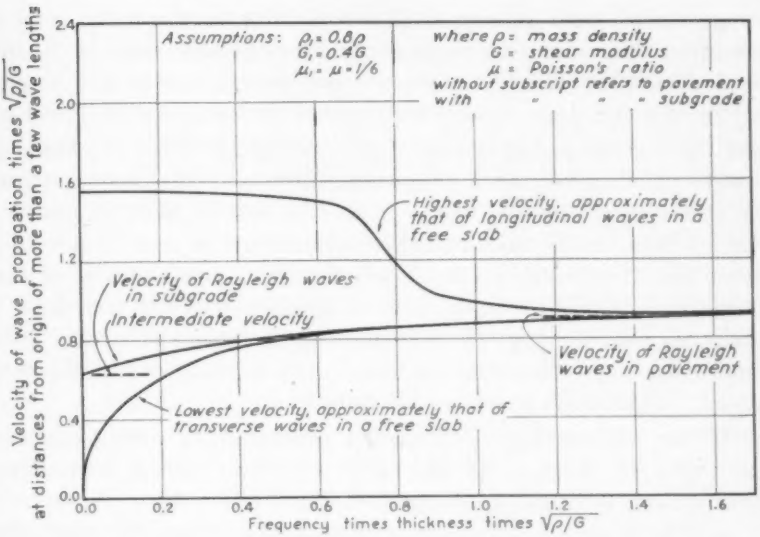
Assumptions: $\rho_1 = 0.8\rho$
$G_1 = 0.4G$
$\mu_1 = \mu = 1/6$
where ρ = mass density
G = shear modulus
μ = Poisson's ratio
without subscript refers to pavement
with " " " subgrade
Highest velocity, approximately that of longitudinal waves in a free slab
Velocity of Rayleigh waves in subgrade
Intermediate velocity
Velocity of Rayleigh waves in pavement
Lowest velocity, approximately that of transverse waves in a free slab
Velocity of wave propagation times $\sqrt{\rho/G}$ at distances from origin of more than a few wave lengths
Frequency times thickness times $\sqrt{\rho/G}$
0.0
0.4
0.8
1.2
1.6
2.0
2.4
0.2
0.6
1.0
1.4

Fig. 2—Effect of frequency of vibration, thickness of pavement, and properties of subgrade on velocity of propagation of waves

These two frequency equations are readily obtained from Equation A by setting G_1 equal to zero. When this is done, either Equation L or T must be satisfied in order that a solution exist.

No correspondingly simple equation can be written for the approximate determination of the intermediate velocity. However, at low frequencies it is approximately the velocity of Rayleigh waves in the subgrade; at high frequencies it is approximately the velocity of Rayleigh waves in the pavement; and at intermediate frequencies it lies between these two limiting velocities. Therefore, a knowledge of these two limiting velocities may be all that is necessary for practical purposes. The relations are:

For Rayleigh waves in the subgrade

$$\left(\frac{2 - b_1^2}{2}\right)^2 = \sqrt{(1 - a_1^2)\,(1 - b_1^2)} \qquad (R_1)$$

and for Rayleigh waves in the pavement

$$\left(\frac{2 - b^2}{2}\right)^2 = k \qquad (R)$$

The foregoing analysis is based upon the assumption that the properties of the subgrade do not vary with depth. The properties of the subgrade, of course, do vary with depth; but, because of dampening, the properties at greater depths probably have no appreciable effect on the wave. It was shown by Love[10] that the velocity of Rayleigh

waves decreased with frequency if the rigidity of the earth increased with depth. Therefore, one might expect in an actual test that the intermediate velocity would first decrease and later increase with increase of frequency rather than exhibit a continual increase as shown by Fig. 2.

Since for a given pavement on a given subgrade there are three possible wave velocities for each frequency, there may be some question as to the situation in a given test. It should also be kept in mind that for each of these wave velocities the motion may be that corresponding to either the H-solution or the V-solution or a combination of them. Final conclusions will probably have to await a study of test data, but it is believed that the type of vibration can be largely controlled by the manner of driving. This belief is based upon a study of the theoretical equations. This study indicates the following:

(1) If the driving force is directed horizontally, waves of the H-solution with the highest of the three velocities should predominate. This conclusion is based upon the fact that in this case points on the line $r = 0$ have only a horizontal motion and of relatively high amplitude. Relatively large motion of the point of application of the driving force and in the direction of the driving force is necessary for large energy input.

(2) If the horizontally directed force is not applied near the central plane of the slab, waves of the lowest of the three velocities will also be produced. These waves will also be of the H-solution. These lower-velocity waves should be produced to the exclusion of the waves of the higher velocity if the driver produces a couple rather than a resultant force. It is believed that with the driving force applied horizontally at the top of the pavement, waves of the highest rather than the lowest velocity will predominate.

(3) If the driving force is directed vertically, waves of the V-solution with the lowest of the three velocities should predominate. This type gives the largest relative vertical motion for points on the line $r = 0$.

(4) Waves of the intermediate velocity should be produced in all cases. Since their velocity is largely controlled by the properties of the subgrade, it is believed that their relative amplitude will be largely controlled by the dampening properties of the subgrade. Perhaps these waves will be most pronounced at low frequencies. It is believed that a rotary driving force such as would be produced by an unbalanced rotating mass would be most favorable for the production of these waves. That is, forces which produced motions for both H- and V-solutions out of phase with each other are believed to be best.

(5) Waves of the V-solution and of the highest of the three velocities could be produced by a pressure bulb inserted at the center of the pavement if the pressure fluctuated periodically.

It should be remembered that the velocities mentioned above are the limiting velocities which are approached asymptotically as the waves get farther from the source. Since velocity varies with distance from the source, the nodes are non-uniformly spaced, especially near the origin, and spaced differently for the different types of vibration. Moreover, in vibration according to the H-solution the character of the motion and the spacing of the nodes are different in the direction of the driving force, north-south directions in Fig. 1, from what they are in a direction at right angles to the driving force, east-west in Fig. 1. The location of the nodes will also depend on which displacements, horizontal or vertical, are being detected.

The table below giving $1/\pi$ times the roots of the Bessel functions involved should be helpful because the relative distances of the nodes from the origin should correspond to the tabular values given. For example, as is evident from the equations for displacement, the vertical displacement in the V-solution is zero at every distance r from the origin for which $J_o(\alpha r)$ is zero.

$1/\pi$ Times the Roots of Bessel Functions

Order of the Root	$J_o(x)$	$J_1(x)$	$\left[J_o(x) - \frac{J_1(x)}{x}\right]$
0	—	0	—
1	0.7655	1.220	0.5860
2	1.757	2.233	1.696
3	2.755	3.238	2.717
4	3.754	4.241	3.726
5	4.753	5.243	4.731
n*	$n - 0.25$	$n + 0.25$	$n - 0.25$

*Where n is a large number.

As is evident from an examination of the equations for displacements, the amplitude of motion of the antinode decreases with distance from the origin. The rate of decrease depends on the angle Θ for waves of the H-solution: for example, *in the direction* of the driving force the amplitude is approximately inversely proportional to the square root of the distance from the origin; *at right angles* to the driving force the amplitude is approximately inversely proportional to the three-halves power of the distance. In the V-solution the motion of the antinodes is approximately inversely proportional to the square root of the distance away from the origin *in any direction*. These facts should be helpful in interpreting results.

Based upon the foregoing theoretical analysis the following experimental procedure with equipment such as that described by Long et al. [6] is recommended.

1. Place driver so as to produce a vertical driving force to the top of the pavement slab.

2. With input to driver set at a given frequency determine location of nodes by moving "pick-up" on radial lines from the driver and noting phase shift and magnitude of response on oscilloscope. (One pair of plates of the cathode ray oscilloscope will be connected to the pick-up circuit and the other pair will be connected to the driver circuit.)

3. Repeat the test at various frequencies.

4. Either repeat the above with driver set so as to produce horizontal motion or use the Long et al. method of determining the velocity of longitudinal waves.

5. Plot velocity of waves versus frequency and compare with curves of Fig. 2.

6. Determine proper values of G/ρ and thickness of slab that will give best agreement with theoretical curves.

7. If data on the properties of the subgrade are desired and the waves of intermediate velocity have not been detected and their velocity determined, use a mechanical driver of relatively low frequency. This should produce the waves of intermediate velocity and of sufficient relative amplitude for detection.

Example

Suppose that at 1000 cycles per second the distances between two nodes (other than the first two nodes) with the driving force acting first vertically and then horizontally are 2.25 ft. and 7.00 ft., respectively. The corresponding velocities will be 4,500 ft. per sec. and 14,000 ft. per sec. ($V = 2l$ times frequency) and the ratio of the velocities will be 4500/14,000 = 0.321. Assume the lower velocity to be that of transverse waves and the higher velocity to be that of longitudinal waves. An examination of Fig. 2 shows that the velocities of these waves have a ratio of 0.321 at a value of about 0.115 for the abscissas and that the corresponding values of the ordinates are approximately 1.55 and 0.50. This means that

$$\sqrt{G/\rho} = \frac{14{,}000}{1.55} = 9{,}030 \text{ ft/sec}$$

and

$$\text{thickness} = \frac{9{,}030 \times 0.115}{1{,}000} = 1.04 \text{ ft.}$$

If density of pavement is 150 lb. per ft^3, then $\rho = 4.66 \frac{\text{lb.sec}^2}{\text{ft}}$

and $G = 4.66(9030)^2 = 380 \times 10^6$ lb. per ft^2 $= 2.64 \times 10^6$ psi

If $\mu = 1/6$, then $E = 14/6 \times 2.64 \times 10^6 = 6.15 \times 10^6$ psi

A test at another frequency, say 2000 cycles per second, should result in approximately the same values for thickness and moduli for the pavement. If radically different values are obtained, then the waves have not been correctly identified. One velocity in at least one of the tests might have been the intermediate velocity.

If the intermediate velocities are not in accord with the middle curve of Fig. 2, then other curves based on other assumptions should be prepared for these velocities.

All of the above discussion has pertained to sustained vibration (stationary waves). Sustained vibration can be considered as the result of two equal continuous wave-trains (progressive waves) traveling in opposite directions. An adequate treatment of a single, impact-generated wave-train of finite length traveling away from a source is beyond the scope of this paper. Because of the finite length of the train and because many different frequencies are usually represented, not all parts travel at the same velocity and the wave-form changes as the wave proceeds. However, of practical importance is the fact that the higher velocity (upper curve, Fig. 2) is almost independent of frequency for low frequencies. Therefore, a wave-train of longitudinal waves of low frequency can be propagated with relatively little change of wave-form. It is probably because of this fact that the velocities of "longitudinal" waves in pavements have been determined successfully by measuring the time required for a short wave-train of longitudinal waves to travel between two points, the short wave-train being produced by an impact.[6]

The effects of variations in physical properties of the subgrade at different depths below the pavement, and the significance of the fact that the pavement does not extend indefinitely in a horizontal direction have not been considered. However, vibration of appreciable amplitude will probably not extend very far from the source either down into the subgrade or horizontally in the pavement owing to the effect of internal friction, especially at high frequencies. It is therefore believed that only the pavement within a few feet of the source and only the material immediately below the pavement will have an appreciable effect upon the wave velocity near the source when the frequency is relatively high.

If the character of the subgrade at considerable depth is desired, relatively low frequencies would be required. The above analysis may be inadequate for a study of the variation in properties of the subgrade with depth since no provision was made for such variation in the equation.

SUMMARY

Equations are derived for the combined vibration of pavement and subgrade. Numerical solution of the frequency equation shows that for a given slab on a given subgrade three wave velocities are possible for each frequency of vibration. The highest of these velocities is almost the same as that of longitudinal vibration of a free slab; the lowest, almost the same as that of transverse (flexural) vibration of a free slab; and the intermediate, somewhere between the velocity of Rayleigh waves in the uncovered subgrade and the velocity of Rayleigh waves in the pavement.

Suggestions are given for the production of any one of the three possible velocities to the virtual exclusion of the other two.

NOTATION (PARTIAL LIST)

r, Θ, z = cylindrical coordinates

u, v, w = displacements in r, Θ, z directions, respectively, of the point (r, Θ, z) in the pavement

u_1, v_1, w_1 = displacements in r, Θ, z directions, respectively, of the point (r, Θ, z) in the subgrade

t = time

ρ, ρ_1 = mass densities of pavement and subgrade, respectively

$$\lambda = \frac{E}{(1 + \mu)(1 - 2\mu)}$$

$$\lambda_1 = \frac{E_1}{(1 + \mu_1)(1 - 2\mu_1)}$$

E, E_1 = Young's modulus for pavement and for subgrade, respectively

μ, μ_1 = Poisson's ratio for pavement and subgrade, respectively

G, G_1 = Shear modulus for pavement and subgrade, respectively

$\sigma_x, \sigma_y, \sigma_z$ = normal stresses

$\tau_{rz}, \tau_{\Theta z}$, etc. = shear stresses

e_x = normal strain in x-direction

$\gamma_{rz}, \gamma_{\Theta z}, \gamma_{xy}$ = shear strains

$p = 2\pi$ times frequency of sustained vibration

$\alpha = p/V = 2\pi/l$ where V is the radial velocity which the wave approaches asymptotically as it gets farther from its source and l is the corresponding wave length

c = half thickness of pavement

$a = \frac{p}{\alpha}\sqrt{\frac{\rho}{\lambda + 2G}}$, ratio of velocity of propagation to velocity of compressional waves in interior of pavement

$b = \frac{p}{\alpha}\sqrt{\frac{\rho}{G}}$, ratio of velocity of propagation to velocity of shear waves in interior of pavement; ordinate of Fig. 2

$m = \alpha\sqrt{1 - a^2}$

$n = \alpha\sqrt{1 - b^2}$

a_1, b_1, m_1, n_1 have the same definitions as $a, b, m,$ and n, respectively, except that those with subscript unity refer to subgrade instead of pavement

$k = \sqrt{(1 - a^2)(1 - b^2)}$

$Re[\ \ ]$ = real part of expression in brackets

J_o, J_1 = Bessel functions of the first kind of order zero and unity, respectively

H_o^1, H_1^1 = Bessel functions of the third kind

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Estimating 28-Day Strength of Concrete from Earlier Strengths—Including the Probable Error of the Estimate*

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Member American Concrete Institute

SYNOPSIS

This paper presents a method for estimating the 28-day strength of concrete from earlier strengths. Using a simple basic formula, its coefficient is adjusted by applying the method of least squares to a small number of data obtained from the mix under consideration. The method is noteworthy because it demonstrates that: only limited data are required for estimating purposes; earlier strengths in weighted combination can be used to estimate 28-day strength with increased accuracy; the formula can be computed accurately with a 10-in. slide rule; and, because it presents a criterion for judging accuracy of estimates.

INTRODUCTION

The writer has had occasion to be impressed with the cost involved in preparing and testing a large number of concrete specimens and, even more so, with the inconvenience and delay in construction caused by having to wait for 28-day results when earlier strengths were on hand. Application of existing formulas for estimating 28-day strengths from earlier results led to the conclusion that these formulas were designed to cover too wide a field. Inasmuch as 28-day concrete strength depends upon a number of variables such as type of mix, cement and aggregates, and methods of curing and testing, these formulas, whose constants were based on the data obtained from a theoretical average concrete, cannot be expected to yield reliable results except, of course, in cases where by chance the concrete under consideration is similar to that on which the constants were based.

The solution was to develop a general formula which would apply to all classes of concrete, and then to adjust the parameters of the formula by applying the method of least squares to a small number of data

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obtained from the specific mix under consideration. Another thought was that the formula had to be practical; that is, capable of accurate computation on a 10-in. slide rule. Finally, a criterion for judging the accuracy of the computations was deemed desirable.

The method which follows is believed to meet these conditions of generality, singularity and practicability. It is to be hoped that its use may lead not only to a saving in the number of specimens required, but also to avoidance of construction delays.

The hardening of concrete proceeds approximately exponentially, but the results of parallel measurements of strength are subject to a large dispersion which makes hazardous any prediction of 28-day strength by exponential extrapolation from earlier strengths. It is practical, however, to express 28-day strength as a linear or non-linear combination of the others. This is done in the ensuing discussion* in which two-parameter formulas are first fitted by least squares to a preliminary series of data; then reduced to one-parameter equations, and used thereafter to estimate 28-day strengths from observed 3- and 7-day strengths of parallel specimens. For different jobs, the values of the parameters will vary appreciably only if the conditions defining parallel specimens vary; otherwise their values should remain fixed.

Parallel specimens are defined here as concrete specimens of approximately identical mix, type of cement, aggregates, specimen dimensions, and curing.

Specimens may no longer be considered parallel when, for instance, a mix is changed from 1:1:3 to 1:3:6; when the cement is changed from Type 1 to High Early Strength; when the coarse aggregates vary from trap rock to crushed limestone, and the fine aggregates from Potomac river sand to Puerto Rico beach sand; when the dimensions of the specimens are changed from 6 x 12-in. to 3 x 6-in. cylinders; and when laboratory curing becomes field curing. On the other hand, water-cement ratio and slump may vary, without affecting parallelity.

1—FORMULAS AND APPLICATION

The method which follows is general, and may be used to estimate the 28-day strengths of concretes and mortars from *any* earlier strengths. The method may also be used to compute flexural and tensile as well as compressive strengths.

Let t = observed 3-day strength, psi.
s = observed 7-day strength, psi.
f = observed 28-day strength, psi.
f_e = predicted 28-day strength, psi.

*See part 2.—Development of Theory

We define the combination c as

$$c = \frac{t + 2s}{3} \quad \ldots\ldots (1)$$

and seek to fit the data by means of the equation

$$f_e = Kd \quad \ldots\ldots (2)$$

Equation (1) expresses the average relative weight of the 3-day strength t as compared to the 7-day strength s in estimating the predicted 28-day strength f_e. The writer has found that in cases where the 7-day strength could be used to predict the 28-day strength within a maximum probable error of, say 240 psi., the prediction from the 3-day strength resulted in a maximum probable error of 480 psi. This, therefore, is our justification for assigning a weight of 2 to the 7-day strength when using the 3-day and 7-day strengths in combination to estimate the 28-day strengths.

The coefficient K in equation (2) is determined by preliminary computations on data obtained from a specified number of parallel specimens from a given job. The preliminary data provide the equations

$$\begin{aligned} f_1 &= Kd_1 \\ &\cdot\ \cdot \\ &\cdot\ \cdot \\ &\cdot\ \cdot \\ f_n &= Kd_n \end{aligned} \quad \ldots\ldots (3)$$

in which n is the set number.

The least squares solution of equation (3) is

$$K = \frac{\text{Summation } (fd)}{\text{Summation } (d^2)} \quad \ldots\ldots (4)$$

Let us assume that*

$$d = 1 + 0.00221\, c \quad \ldots\ldots (5)$$

for plotted data which exhibit substantially no curvature,

$$d = c + 25.9\, c^{\frac{1}{2}} \quad \ldots\ldots (6)$$

for plotted data which exhibit only slight curvature, and

$$d = c - 0.0000692\, c^2 \quad \ldots\ldots (7)$$

for plotted data which exhibit appreciable curvature.

In choosing an empirical formula to represent specific data, we first plot the given data by taking the observed 3-day t, 7-day s, or their combination c values as abscissas, and the corresponding observed 28-day f strengths as ordinates, and pass a curve through the plotted points. The features of such a curve are best brought out if the scales of the two axes are so chosen that the curve drawn makes approximately equal angles with them. This may be accomplished as follows: If the maxi-

*See part 2.—Development of Theory

mum required value along the ordinate is, say 6000 psi., and the ordinate scale chosen is such that 1-in. equals 1000 psi., and the maximum value along the abscissa axis is, say 4000 psi.; then the abscissa scale should be 1.5-in. equals 1000 psi. With the data plotted in this manner, the curve will show to the eye how f varies with t, s or c. If the plot develops as substantially a straight line, equations (1), (2), (4), and (5) are used; if slight curvature is exhibited, equations (1), (2), (4) and (6) may fit best; and if the plot shows appreciable curvature, equations (1), (2), (4) and (7) should be tried.

Exactly which set of formulas will apply best to a given set of data, so as to fit them well, and at the same time be of use in prediction, can be determined only by trial. In evaluating results obtained from the three sets of formulas given above, that one is the best which yields the smallest value of maximum probable error. Finally, if the three sets of equations furnish substantially the same maximum probable error, that one is the best which is the simplest to compute.

However, unless the curvature is very marked, the writer is inclined to believe that the straight line graph represented by equations (1), (2), (4) and (5) will provide substantially as much accuracy for estimating purposes as the other curves when dealing with a material as heterogeneous as concrete. In addition, it is the simplest to compute.

Maximum probable error

The probable error is a function of the *root-mean-square* (RMS) departure of the predicted 28-day strength f_e from the observed 28-day strength f. As used here, the term *maximum probable error* (PE_{max}) is defined to be that value of the probable error which will not be exceeded by the $(f - f_e)$ departures of 90 out of 100 parallel specimens. We find* from the standard probability curve that for such a probability ($P = 0.90$)

$$PE_{max} = 1.645 \left[\frac{N + 2}{N - 2}\right]^{1/2} RMS \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where N represents the total number of sets of parallel specimens used as a base.

Note that, because of the approximate nature of equation (8), it is not considered trustworthy for values of N less than 6.

The question naturally arises as to how many sets of parallel specimens are required as a base in order to obtain reliable predictions of 28-day strength from earlier strengths. It might be thought that if the coefficient K in equation (2) were based on extensive data and long experience, predictions of 28-day strength would be much more accurate than with a value of K obtained from a limited series of data.

*See part 2.—Development of Theory

The answer to this question* is that no value of K, however painstakingly chosen, can reduce the maximum probable error below a certain minimum, which value is closely approached by the use of a K obtained from a limited series of data. This is indeed a fortunate and significant conclusion.

Let us consider what this means. If, for example, we are working with a concrete mix which develops a 28-day strength of 3000 psi., and with a value of K based on most extensive data and long experience ($N = \infty$), the 28-day strength might possibly be predicted within an indicated maximum probable error of, say 240 psi. On the other hand, if K were based on $N = 10$ or $N = 6$ specimens, the maximum probable errors would be 264 or 276 psi., respectively. This means that the maximum probable error for $N = \infty$, $N = 10$ and $N = 6$ is 8.0 percent, 8.8 per cent and 9.2 percent of 3000 psi., respectively.

Here, therefore, is our justification for using only a relatively small number of specimens for predicting 28-day strength from earlier strengths.

We are now ready to illustrate the use of the foregoing equations on specific test data.

Problem 1.—Prediction of 28-day strength from observed 3- and 7-day strengths combined.

G. B. Sheldon, Jr., construction engineer, Public Buildings Administration, Washington, D. C., furnished the data shown in Table 1, concerning 15 sets of 3-, 7-, and 28-day (6 x 12-in.) concrete cylinders made during construction of the Hydrographic Building at Suitland, Maryland, from March 10 to May 22, 1942.

The materials used per cubic yard of concrete were: 517-lb. of Type I portland cement, 1430-lb. of bank-run sand, and 1870-lb. of ¾-in. bank-run gravel. The cylinders remained on the site for the first 24 hours, and were then taken to the Public Building Administration laboratory, and cured in a saturated atmosphere at 70 F. until tested.

Slumps ranged from 3½-in. to 6½-in. Average temperatures during the first 24 hours varied from 47 to 68 F. Also, the cylinder sets marked "v" were vacuum processed. The large range of strengths noted is probably due to the combined influence of variations in water content, first day curing temperatures, and vacuum processing.

The test results were arranged from low to high on the basis of 3-day tests. Five representative sets were then selected by Mr. Sheldon for purposes of prediction. The latter are identified as set numbers 11 to 15.

We now enter the pertinent data of cylinder sets 1 to 10 in Table 2 and compute the values of c. We note that the maximum value to be plotted along the ordinate is 4250, and that the maximum value along the abscissa is 3100. We therefore select an ordinate scale of 1-in. equals 1000 psi., and an abscissa scale of 4250/3100, or 1.4-in. equals 1000 psi. We then plot values of c of sets 1 to 10 against the corresponding values of f, in Fig. 1. Since the graph through the 10 points exhibits substantially

*See part 2.—Development of Theory

TABLE 1—COMPRESSIVE STRENGTHS OF 6 x 12-IN. CYLINDERS

Suitland Test Results

Set Number (n)	3-Day (t) psi.	7-Day (s) psi.	28-Day (f) psi.
1	980	1520	2230
2	1200	1720	2320
3	1240	2110	2660
4	1430	1760	2640
5v	1720	2540	3340
6v	1780	2250	3240
7	1980	2900	3900
8	2060	2900	3820
9	2250	2750	3550
10v	2730	3290	4250
11	1240	1920	2730
12v	1540	2040	2690
13	1910	2500	3470
14v	2220	2710	3800
15v	3170	4560	5560

TABLE 2—CALCULATION OF K FROM OBSERVED 3, 7 AND 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 1 TO 10

Set No. n	3-Day t psi.	7-Day s psi.	28-Day f psi.	c psi.	d	d^2	df
1	980	1520	2230	1340	3.96	15.7	8830
2	1200	1720	2320	1550	4.43	19.6	10280
3	1240	2110	2660	1820	5.02	25.2	13350
4	1430	1760	2640	1650	4.65	21.6	12280
5	1720	2540	3340	2270	6.02	36.2	20100
6	1780	2250	3240	2090	5.62	31.6	18210
7	1980	2900	3900	2590	6.72	45.2	26200
8	2060	2900	3820	2620	6.79	46.1	25900
9	2250	2750	3550	2580	6.70	44.9	23800
10	2730	3290	4250	3100	7.85	61.6	33400
$f_c = Kd;\ d = 1 + 0.00221c$				$c = \frac{t + 2s;}{3}$		347.7	192350

$$K = \frac{192350}{347.7} = 554$$

no curvature, we choose to work with equations (1), (2), (4) and (5). Using these to compute and complete the entries in Table 2, we find that $K = 554$.

In Table 2A, we estimate the 28-day strengths of cylinder sets 11 to 15 from their respective observed 3- and 7-day strengths. The results and departures are as in Table 2B.

To determine the maximum probable error, we first calculate the value of RMS from the observed 3-, 7-, and 28-day strengths of sets 1 to 10. This is done in Table 3, with RMS = 112 psi. Then, by equation (8), with $N = 10$, the value of the maximum probable error is found to be

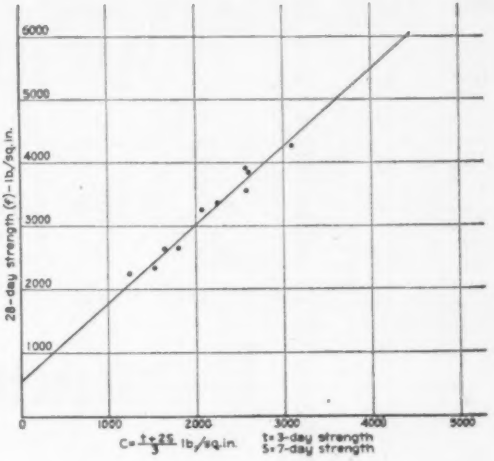


Fig. 1 (Problem 1)—Weighted 3- and 7-day values (c) plotted against 28-day strengths (f) to determine shape of curve through points.

TABLE 2A—CALCULATION OF 28-DAY COMPRESSIVE STRENGTH OF CYLINDER SETS 11 TO 15 FROM OBSERVED 3- AND 7-DAY STRENGTHS

Set No. n	3-Day t psi.	7-Day s psi.	c psi.	d	Predicted 28-Day f_e psi.
11	1240	1920	1690	4.74	2630
12	1540	2040	1870	5.13	2840
13	1910	2500	2300	6.08	3370
14	2220	2710	2550	6.64	3680
15	3170	4560	4100	10.06	5570

$$f_e = Kd; \quad K = 554; \quad d = 1 + 0.00221c; \quad c = \frac{t + 2s}{3}$$

$PE_{max} = 225$ psi

In the prediction of the 28-day strengths of parallel specimens from combined 3- and 7-day strengths, it is probable, therefore, that, in the case of 90 out of 100 cylinders, the individual errors will not exceed 225 psi. Examination of the errors ($f - f_e$ values) in sets 1 to 15 supports this conclusion, all of the departures being smaller than PE_{max}, with the largest, 180 psi occurring in set 7.

TABLE 2B

n	Observed 28-day Strength, f psi.	Predicted 28-day Strength, f_e psi.	Departure $f - f_e$ psi.
11	2730	2630	+ 100
12	2690	2840	— 150
13	3470	3370	+ 100
14	3800	3680	+ 120
15	5560	5570	+ 10

TABLE 3—CALCULATIONS OF RMS FROM OBSERVED 3, 7 AND 28-DAY STRENGTHS OF CYLINDER SETS 1 TO 10.

Set No. n	3-Day t psi.	7-Day s psi.	c psi.	d	Predicted 28-Day f_e psi.	Observed 28-Day f psi.	$f - f_e$ psi.	$(f - f_e)^2$
1	980	1520	1340	3.96	2190	2230	+ 40	1600
2	1200	1720	1550	4.43	2450	2320	−130	16900
3	1240	2110	1820	5.02	2780	2660	−120	14400
4	1430	1760	1650	4.65	2580	2640	+ 60	3600
5	1720	2540	2270	6.02	3340	3340	0	0
6	1780	2250	2090	5.62	3110	3240	+130	16900
7	1980	2900	2590	6.72	3720	3900	+180	32400
8	2060	2900	2620	6.79	3760	3820	+ 60	3600
9	2250	2750	2580	6.70	3710	3550	−160	25600
10	2730	3290	3100	7.85	4350	4250	−100	10000

$f_e = Kd$; $d = 1 + 0.00221c$; $c = \frac{t + 2s}{3}$

$K = 554$

Sum = 125000
Mean = 12500
RMS = 112

TABLE 4—CALCULATION OF K FROM OBSERVED 3 AND 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 1 TO 10

Set No. n	3-Day t psi.	28-Day f psi.	$t^{1/2}$	d	d^2	df
1	980	2230	31.3	1790	320×10^4	399×10^4
2	1200	2320	34.6	2100	441	487
3	1240	2660	35.2	2150	462	572
4	1430	2640	37.8	2410	581	636
5	1720	3340	41.5	2800	784	935
6	1780	3240	42.2	2870	824	930
7	1980	3900	44.5	3130	980	1221
8	2060	3820	45.4	3240	1050	1238
9	2250	3550	47.4	3480	1211	1235
10	2730	4250	52.2	4080	1665	1734

$f_e = Kd$; $d = c + 25.9c^{1/2}$; $c = t$

8318×10^4 9387×10^4

$$K = \frac{9387 \times 10^4}{8318 \times 10^4} = 1.13$$

TABLE 4A—CALCULATION OF 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 11 TO 15 FROM OBSERVED 3-DAY STRENGTHS

Set No. n	3-Day t psi.	$t^{1/2}$	d	Predicted 28-Day f_e psi.
11	1240	35.2	2150	2430
12	1540	39.2	2560	2890
13	1910	43.7	3040	3440
14	2220	47.1	3440	3890
15	3170	56.3	4630	5230

$f_e = Kd$; $K = 1.13$; $d = c + 25.9c^{1/2}$; $c = t$

Problem 2.—Prediction of 28-day strength from observed 3-day strength.

For some purposes, for example in connection with the use of high early strength cement, or the vacuum concrete process, it may suffice to make an estimate of the 28-day strength based only on the observed 3-day strength, t. Utilizing the cylinder sets previously given in Table 1, we enter the pertinent data in Table 4 and plot Fig. 2. Since the graph through the 10 points exhibits a slight curvature, we choose to work with equations (1), (2), (4) and (6). We note, however, that in equation (1), $c = t$; therefore, t is used throughout instead of c.

Using the above equations to compute the entries in Table 4, we find that $K = 1.13$.

In Table 4A, we estimate the 28-day strengths of cylinder sets 11 to 15 from their respective observed 3-day strengths. The results and departures are as in Table 4B.

To determine the maximum probable error, we calculate the value of RMS from the observed 3- and 28-day strengths of sets 1 to 10. This is done in Table 5, with RMS = 238 psi. Then, by equation (8), with $N = 10$, the maximum probable error is found to be

$$PE_{max} = 479 \text{ psi.}$$

Examination of the $(f - f_e)$ departures in sets 1 to 15 shows that the largest, 380 psi, occurs in set 9.

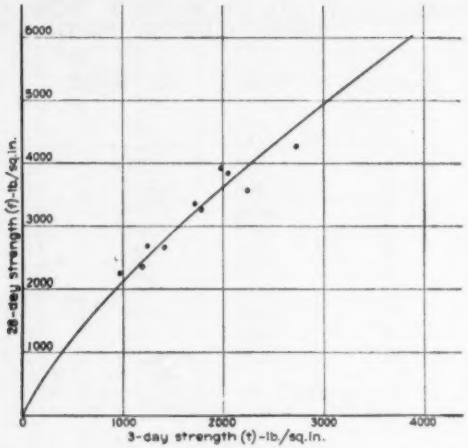


Fig. 2 (Problem 2)—3-day strengths (t) plotted against 28-day strengths (f) to determine shape of curve through points.

TABLE 4B

n	Observed 28-day Strength, f psi.	Predicted 28-day Strength, f_e psi.	Departure $f - f_e$ psi.
11	2730	2430	+ 300
12	2690	2890	− 200
13	3470	3440	+ 30
14	3800	3890	− 90
15	5560	5230	+ 330

TABLE 5—CALCULATION OF RMS FROM OBSERVED 3 AND 28-DAY STRENGTHS OF CYLINDER SETS 1 TO 10

n	d	f_e	f	$(f-f_e)$	$(f-f_e)^2$
1	1790	2020	2230	+ 210	441 00
2	2100	2370	2320	− 50	25 00
3	2150	2430	2660	+ 230	529 00
4	2410	2720	2640	− 80	64 00
5	2800	3160	3340	+ 180	324 00
6	2870	3240	3240	0	0
7	3130	3540	3900	+ 360	1296 00
8	3240	3660	3820	+ 160	256 00
9	3480	3930	3550	− 380	1444 00
10	4080	4610	4250	− 360	1296 00

$f_e = Kd \quad K = 1.13$

$d = c + 25.9c^{1/2}$

$c = t$

Sum = 5675 00
Mean = 567 50
RMS = 238

Problem No. 3—Prediction of 28-day strength from observed 7-day strength.

On most jobs it may suffice to have an estimate of the 28-day strength based only on the observed 7-day strength, s. Using the cylinder sets previously given in Table 1,

TABLE 6—CALCULATION OF K FROM OBSERVED 7 AND 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 1 TO 10

Set No. n	7-Day s psi.	28-Day f psi.	d	d^2	df
1	1520	2230	4.36	19.0	9720
2	1720	2320	4.80	23.0	11140
3	2110	2660	5.66	32.1	15060
4	1760	2640	4.89	23.9	12910
5	2540	3340	6.61	43.7	22080
6	2250	3240	5.97	35.7	19340
7	2900	3900	7.41	54.9	28900
8	2900	3820	7.41	54.9	28300
9	2750	3550	7.08	50.1	25100
10	3290	4250	8.27	68.4	35100
$f_e = Kd$; $d = 1 + 0.00221c$; $c = s$; $K = \frac{207650}{405.7} = 512$				405.7	207650

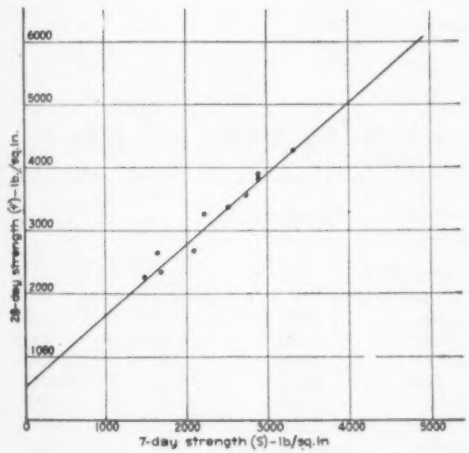


Fig. 3 (Problem 3)—7-day strengths (s) plotted against 28-day strengths (f) to determine shape of curve through points

we enter the pertinent data in Table 6 and plot Fig. 3. Since the graph through the 10 points exhibits substantially no curvature, we again choose to work with equations (1), (2), (4) and (5). We note, however, that in equation (1), $c = s$; therefore, s is used throughout instead of c.

Using the above equations to compute the entries in Table 6, we find that $K = 512$.

In Table 6A, we estimate the 28-day strengths of cylinder sets 11 to 15 from their respective observed 7-day strengths. The results and departures are as in Table 6B.

To determine the maximum probable error, we calculate the value of RMS from the observed 7- and 28-day strengths of sets 1 to 10. This is done in Table 7, with RMS = 121 psi. Then, by equation (8), with $N = 10$, the value of the maximum probable error is found to be

$$PE_{max} = 243 \text{ psi.}$$

TABLE 6A—CALCULATION OF 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 11 TO 15 FROM OBSERVED 7-DAY STRENGTHS

Set No. n	7-Day s psi.	d	Predicted 28-Day f_s psi.
11	1920	5.24	2680
12	2040	5.51	2820
13	2500	6.53	3340
14	2710	6.99	3580
15	4560	11.08	5670

$f_s = Kd$; $K = 512$; $d = 1 + 0.00221c$; $c = s$

TABLE 6B

n	Observed 28-day Strength, f psi.	Predicted 28-day Strength, f_s psi.	Departure $f - f_s$ psi.
11	2730	2680	+ 50
12	2690	2820	− 130
13	3470	3340	+ 130
14	3800	3580	+ 220
15	5560	5670	− 110

TABLE 7—CALCULATION OF RMS FROM OBSERVED 7 AND 28-DAY STRENGTHS OF CYLINDER SETS 1 TO 10

Set No. n	7-Day s psi.	d	Predicted 28-Day f_s psi.	Observed 28-Day f psi.	$f - f_s$	$(f - f_s)^2$
1	1520	4.36	2230	2230	0	0
2	1720	4.80	2460	2320	−140	19600
3	2110	5.66	2900	2660	−240	57600
4	1760	4.89	2500	2640	+140	19600
5	2540	6.61	3380	3340	− 40	1600
6	2250	5.97	3060	3240	+180	32400
7	2900	7.41	3800	3900	+100	10000
8	2900	7.41	3800	3820	+ 20	400
9	2750	7.08	3620	3550	− 70	4900
10	3290	8.27	4230	4250	− 20	400

$f_s = Kd$; $d = 1 + 0.00221c$; $c = s$; $K = 512$ Sum = 146500
Mean = 14650
RMS = 121

Examination of the $(f - f_s)$ departures in sets 1 to 15 shows that the largest, 240 psi, occurs in set 3.

Problem No. 4.—Prediction of 28-day strength from observed 7-day strength of nodulite concrete.

H. J. McGillivray, Chief of the Concrete Control and Testing Division, Hooker's Point Shipyard, Tampa, Fla., furnished the data given in Table 8, concerning 15 sets

of 7- and 28-day (6 x 12-in.) concrete cylinders made during the construction of 24 self-propelled reinforced-concrete cargo vessels from June, 1943 to Sept., 1944.

The materials used per cubic yard of concrete were as follows: 9.2 bags of type 2 portland cement; 1370-lb. of coarse, medium and fine Nodulite, ranging in size from ½ inch to dust; 410-lb. of Lake Wales sand, ranging in size from ⅛ inch to dust; and 4.6-lb. of Plastiment. The average fineness modulus of the combined Nodulite sizes was 4.28. The fineness modulus for the sand was 1.98. Water was used in such quantity as to obtain slumps of from 3½ to 4½ inches in the mix.

Nodulite, a lightweight aggregate developed for use in the concrete ships built at Tampa, is made of Fuller's Earth, molded and burned in roughly spherical shapes in a plant at Ellenton, Fla., constructed for this purpose.

All cylinders were made in the concrete laboratory at the Yard from the field mix as delivered in buggies from the transit-mix trucks. The cylinders were cured in a fog-room at 70 F. until tested.

TABLE 8—COMPRESSIVE STRENGTHS OF 6 X 12-IN. CYLINDERS

Hooker's Point Shipyard Test Results

Set Number (n)	7-Day (s) psi.	28-Day (f) psi.
101	3460	5150
102	3890	5480
103	4010	5410
104	4200	5530
105	4200	5960
106	4390	6410
107	4510	6550
108	4790	6010
109	5090	6680
110	5170	6750
111	3680	5220
112	4330	6000
113	4820	6220
114	4970	6010
115	5000	6660

The test results in Table 8 were arranged from low to high on the basis of the 7-day tests, and five sets were then selected for purposes of prediction. The latter are identified as set numbers 111 to 115.

Utilizing the cylinder sets 101 to 110 given in Table 8, we enter the pertinent data in Table 9, and plot Fig. 4. Since the graph through the 10 points exhibits appreciable curvature, we choose to work with equations (1), (2), (4) and (7). We note, however, that $c = s$, in equation (1); hence, s is used throughout instead of c.

Using the above equations to complete the entries in Table 9, we find that $K = 1.98$.

In Table 9A, we estimate the 28-day strengths of cylinder sets 111 to 115 from their respective observed 7-day strengths. The results and departures are as in Table 9B.

To determine the maximum probable error, we calculate the value of RMS from the observed 7- and 28-day strengths of sets 101 to 110. This is done in Table 10, with RMS = 275 psi. Then, by equation (8), with $N = 10$, the maximum probable error is found to be

$$PE_{max} = 554 \text{ psi.}$$

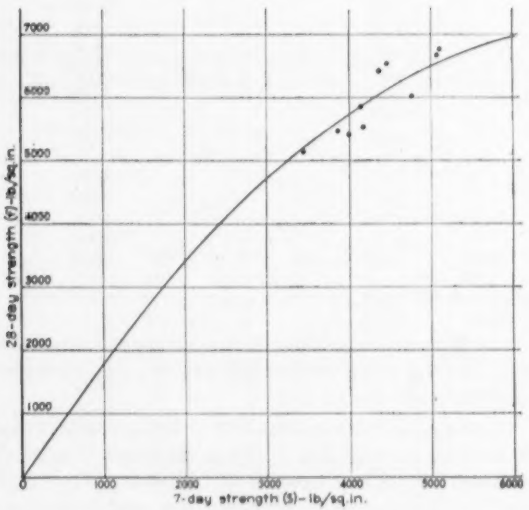
TABLE 9—CALCULATION OF K FROM OBSERVED 7 AND 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 101 TO 110

Set No. n	7-Day s psi.	28-Day f psi.	s^2	d	d^2	df
101	3460	5150	119×10^6	2640	697×10^4	1359×10^4
102	3890	5480	151	2840	807	1556
103	4010	5410	161	2900	841	1569
104	4200	5530	176	2980	888	1648
105	4200	5960	176	2980	888	1776
106	4390	6410	193	3050	930	1955
107	4510	6550	203	3100	961	2030
108	4790	6010	229	3210	1030	1929
109	5090	6680	259	3300	1089	2200
110	5170	6750	267	3320	1102	2240
					9233×10^4	18262×10^4

$f_s = Kd$ $\quad d = c - 0.0000692c^2$ $\quad c = s$

$$K = \frac{18262 \times 10^4}{9233 \times 10^4} = 1.98$$

Fig. 4 [(Problem 4)—7-day strengths (s) plotted against 28-day strengths (f) to determine shape of curve through points



Examination of the $(f - f_s)$ departures in sets 101 to 115 shows that the largest, 440 psi, occurs in cylinder sets 114.

Recommended procedure

With the mix, type of cement and aggregates, specimen dimensions, curing and testing techniques determined, the following sequence of operations is recommended:

TABLE 9A—CALCULATION OF 28-DAY COMPRESSIVE STRENGTHS OF CYLINDER SETS 111 TO 115 FROM OBSERVED 7-DAY STRENGTHS

Set No. n	7-Day s psi.	s^2	d	Predicted 28-Day Strength, f_e psi.
111	3680	135×10^6	2750	5450
112	4330	187 "	3040	6020
113	4820	232 "	3210	6360
114	4970	247 "	3260	6450
115	5000	250 "	3270	6470

$f_e = Kd$; $K = 1.98$; $d = c - 0.0000692c^2$; $c = s$

TABLE 9B

n	Observed 28-day Strength, f psi.	Predicted 28-day Strength, f_e psi.	Departure $(f - f_e)$ psi.
111	5220	5450	− 230
112	6000	6020	− 20
113	6220	6360	− 140
114	6010	6450	− 440
115	6660	6470	+ 190

TABLE 10—CALCULATION OF RMS FROM OBSERVED 7 AND 28-DAY STRENGTHS OF CYLINDER SETS 101 TO 110

n	d	f_e	f	$f - f_e$	$(f - f_e)^2$
101	2640	5230	5150	− 80	6400
102	2840	5620	5480	− 140	19600
103	2900	5740	5410	− 330	108900
104	2980	5900	5530	− 370	136900
105	2980	5900	5960	+ 60	3600
106	3050	6040	6410	+ 370	136900
107	3100	6140	6550	+ 410	168100
108	3210	6360	6010	− 350	122500
109	3300	6530	6680	+ 150	22500
110	3320	6570	6750	+ 180	32400

$f_e = Kd$ $K = 1.98$
$d = c - 0.0000692c_2$
$c = s$

Sum = 757800
Mean = 75780
RMS = 275

a. Make ten (but not less than six) sets of 3-, and/or 7-day, and 28-day specimens, varying the slump from 0.5 to 8-in., and record the test results.

b. Use the data from the above series to plot earlier strengths against 28-day strengths to proper scales.

c. Draw a curve through the plotted points and then select the appropriate sets of equations as discussed previously. To repeat, the

straight line graph represented by equations (1), (2), (4) and (5) is recommended for practically all cases encountered in practice.

d. Calculate the values of K and PE_{max}.

e. Draw a job curve representing $f_e = Kd$. This will be used thereafter to estimate 28-day strengths from earlier strengths.

f. Parallel to the above-mentioned curve, draw two other dotted curves representing the upper and lower limits defined by PE_{max}. Use these as a guide in evaluating the quality of concrete control on the job.

g. Thereafter, for every 100 specimens made to represent earlier strengths, make 10 specimens to be broken at 28 days. These are to be used as a check on the continuing accuracy of both K and PE_{max}.

2—DEVELOPMENT OF THEORY

Formulas

Case 1.—Plotted data exhibit substantially no curvature.

We seek to fit the data by means of the equation

$$f_e = A + Bc \quad (9)$$

The values of the parameters A and B are to be determined from preliminary measurements on a job. The preliminary data provide the equations

$$\begin{aligned} f_1 &= A + Bc_1 \\ &\cdot \quad \cdot \quad \cdot \\ &\cdot \quad \cdot \quad \cdot \\ &\cdot \quad \cdot \quad \cdot \\ f_n &= A + Bc_n \end{aligned} \quad (10)$$

n being the set number.

The least squares solution of equation (10) is

$$A = \frac{TQ - CP}{NQ - C^2} \quad (11)$$

and

$$B = \frac{NP - CT}{NQ - C^2} \quad (12)$$

into which we have introduced for simplicity the notations

$$\begin{aligned} C &= \text{Summation of } c \\ T &= \text{Summation of } f \\ P &= \text{Summation of } c^2 \\ Q &= \text{Summation of } cf \end{aligned} \quad (13)$$

If we let $A = K$, and $B = MK$, equation (9) becomes

$$f_e = Kd; \text{ where } d = 1 + Mc \quad (14)$$

We have transformed the parameters from A and B to K and M. If now we cease to regard M as an adjustable parameter, but fix it in advance, we will have only K to adjust.

The determination of M should logically be based on comprehensive data and long experience. In lieu thereof, the writer believes that a likely value is $M = 0.00221$. Substituting this value for M in equation (14), we obtain

$$f_e = Kd \quad (2)$$

in which

$$d = 1 + 0.00221\,c \quad (5)$$

Therefore, with M fixed in advance, we have a one-parameter formula, the value of which is readily determined by a short series of data. Thus, at any stage on a given job, we can check the value of K from the last few pours and thereby quickly detect changes in field conditions.

Case 2.—Plotted data exhibit slight curvature.

We seek to fit the data by means of the two-parameter equation

$$f_e = Ac + Bc^{1/2} \quad (15)$$

of which the least squares solution is

$$A = \frac{CP - YZ}{CQ - Z^2} \quad (16)$$

and

$$B = \frac{QY - PZ}{CQ - Z^2} \quad (17)$$

where

C = Summation of c
P = Summation of cf
Q = Summation of c^2
Y = Summation of $c^{1/2}f$
Z = Summation of $c^{1.5}$

By using a technique similar to that shown in Case 1, the writer has determined that the corresponding one-parameter equation is

$$f_e = Kd \quad (2)$$

in which

$$d = c + 25.9c^{1/2} \quad (6)$$

Case 3.—Plotted data exhibit appreciable curvature.

We seek to fit the data by means of the two-parameter equation

$$f_e = Ac + Bc^2 \quad (18)$$

of which the least squares solution is

$$A = \frac{PS - RW}{QS - R^2} \quad (19)$$

and

$$B = \frac{QW - PR}{QS - R^2} \quad (20)$$

where

P = Summation of cf
Q = Summation of c^2
R = Summation of c^3
S = Summation of c^4
W = Summation of c^2f ..... (21)

By using a device similar to that shown in Case 1, the writer has determined that the corresponding one-parameter equation is

$$f_e = Kd \quad \text{..... (2)}$$

in which

$$d = c - 0.0000692c^2 \quad \text{..... (7)}$$

Probable error

The probable error is a function of the standard error (SE), which in turn is simply the *root-mean-square* (RMS) departure of the predicted 28-day strength (f_e) from the observed 28-day strength (f). However, three RMS values must be distinguished:

RMS(N_1), or the RMS for N_1 pours, the values of A and B, or K being obtained directly from the pours.

RMS($N \propto$) or the RMS for an infinitely large number of pours, using *true* values of A and B, or K.

RMS(N_2) or the RMS to be encountered in future work, the values of A and B, or K being based on experience on similar jobs, or on N_2 pours.

For our purposes, the significant RMS is the third, therefore

$$SE = \text{RMS}(N_2) \quad \text{..... (22)}$$

We have available an approximate formula for RMS(N_2) expressed in terms of RMS($N \propto$), or

$$\text{RMS}(N_2) = \left[\frac{N_2 + 2}{N_2}\right]^{1/2} \text{RMS}(N \propto) \quad \text{..... (23)}$$

We also have available an approximate formula for RMS($N \propto$) expressed in terms of RMS(N_1), or

$$\text{RMS}(N \propto) = \left[\frac{N_1}{N_1 - 2}\right]^{1/2} \text{RMS}(N_1) \quad \text{..... (24)}$$

Combining equations (22), (23) and (24) to obtain a general formula, we have

$$SE = \left[\frac{N_2 + 2}{N_2} \times \frac{N_1}{N_1 - 2}\right]^{1/2} \text{RMS}(N_1) \quad \text{..... (25)}$$

We may distinguish two cases of interest: If $N_1 = N_2$, equation (25) becomes

$$SE = \left[\frac{N + 2}{N - 2}\right]^{1/2} \text{RMS}\ (N) \quad \text{..... (26)}$$

and, if $N_2 = \infty$, equation (25) becomes

$$SE_{min} = \left[\frac{N}{N-2}\right]^{1/2} \text{RMS}(N) \quad \ldots\ldots (27)$$

Let us now inquire by how much the accuracy of SE in equation (26) would be increased if it were based on unlimited data and long experience instead of a small number of observations. The answer to this question may be obtained by analyzing the ratio obtained in dividing equation (26) by equation (27), or

$$\frac{SE}{SE_{min}} = \left[\frac{N+2}{N}\right]^{1/2} \quad \ldots\ldots (28)$$

It is of interest to compute this ratio for several values of N. This is done in Table 11. It should be kept in mind that, because of inherent approximations, equation (28) is not considered reliable for values of N less than 6; hence, lower values were not computed.

TABLE 11—INCREASE IN ACCURACY OF STANDARD ERROR WITH INCREASE IN NUMBER OF CYLINDER SETS USED TO COMPUTE A AND B, OR K

Cylinder Sets, N	Error in SE percent
6	15
8	12
10	10
20	5
100	1
Infinity	0

Table 11 informs us that no practical end is served in using a large number of data to evaluate A and B, or K; therefore, the use of more than a limited series of data for this purpose is not indicated.

We now adopt the concept of *maximum probable error* (PE_{max}), and define it as that value of the probable error which will not be exceeded by the individual departures of 90 out of 100 parallel specimens. For such a probability ($P = 0.90$), we find from the standard probability curve that

$$PE_{max} = 1.645\ SE = 1.645 \left[\frac{N+2}{N-2}\right]^{1/2} \text{RMS} \quad \ldots\ldots (8)$$

ACKNOWLEDGMENTS

The writer is particularly grateful to Dr. Archie Blake for assistance in the application of the fundamental theory.

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7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN (Supplement) November 1945

Discussion of a paper by Jacob J. Creskoff:

Estimating 28-Day Strength of Concrete from Earlier Strengths—Including the Probable Error of the Estimate*

By S. P. WING and AUTHOR

By S. P. WING†

The author's method of estimating 28-day concrete strength from tests at earlier ages appears more complicated than it really is. His explanation of the method of least squares for fitting a straight line to data, together with his algebraic expression of the results, tends to obscure the essential simplicity of the treatment. In brief the method consists of four steps: (1) making a pilot test series with varying water-cement ratios; (2) plotting the strengths of 28-day cylinders against those of companion cylinders broken at an earlier age; (3) fitting the data with a straight line passed through the point defined by the averages of the two series; and (4) using for subsequent tests the numerical value of the slope of the line as the coefficient by which an observed departure of an early age test from its mean is to be multiplied to obtain the predicted departure of the 28-day strength from its mean. Experience shows that these coefficients (slopes) vary from about 0.5 to 2.0, the exact value being dependent on the particular job.

Graph A, Fig. A, shows a re-plot of data from the author's Fig. 3 with a straight line fitted through the point defined by the 7- and 28-day average strengths. Its slope is 1.13. Graph B shows a straight line similarly fitted to field control data from another job with tests extending over a 2-year period‡; its slope is 0.65. On the job represented by Graph A, 7-day concrete testing 100 lb. above its average would be expected to yield 28-day concrete testing $1.13 \times 100 = 113$ psi. above the 28-day average. For the job of Graph B, for the same 7-day strength change, the predicted deviation at 28 days would be only 65 psi. These simple prediction coefficients are hidden within the author's values for

*ACI JOURNAL, Apr. 1945; *Proceedings* V. 41, p. 493.
†Civil Engineer, Bureau of Reclamation, Denver, Colo.
‡Each plotted point from averages of the 28- and 7-day cylinders of two batches made 3 days apart. W/C = 0.58.

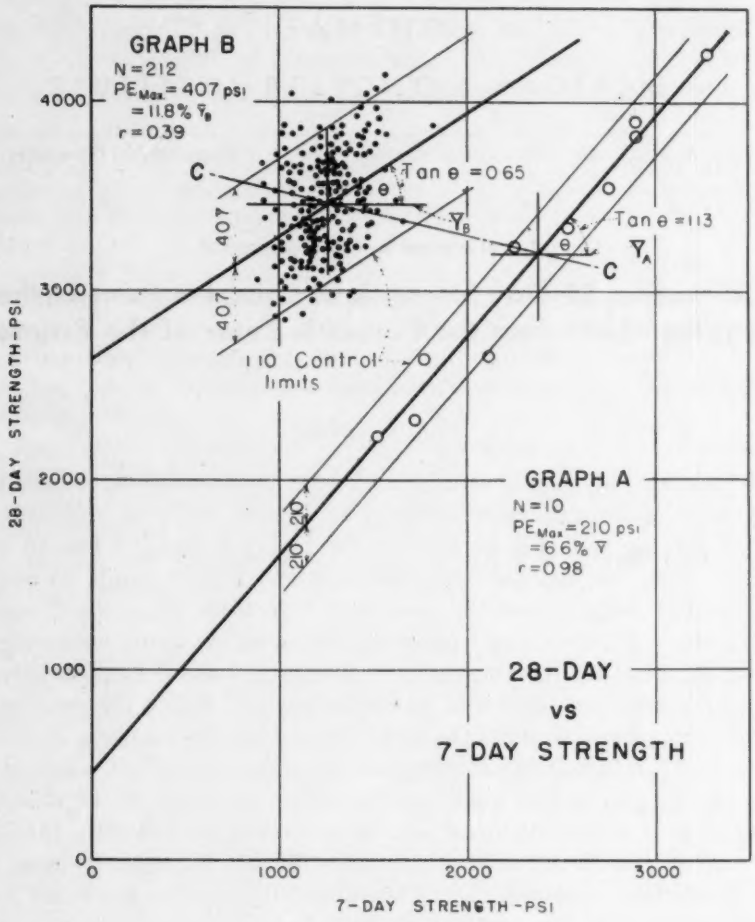

Fig. A

K and for the coefficient of C. The product of these two, taking their values from the author's Table 7, is $512 \times 0.00221 = 1.13$, the slope of the line of Graph A.

Although the paper goes into a number of alternatives for refining this method, such as using a weighted mean of 3- and 7-day strengths as the independent variable, or using a curve to fit the data, it is believed that a study of the results from a large number of tests from any job will result in a plot similar to that of Graph B. With limited data of the type the author presented in Fig. 1-A in which only a single variable such as water-cement ratio controls the variation, a low prediction error of ±6.6 percent (1:10 chance) makes the method look encouraging. But for Graph B, where tests cover a considerable time, the prediction error of

±11.8 per cent is but little less than the error which would be obtained by assuming the average 28-day strength as the predicted value regardless of the indication of a 7-day result.

That the large error in prediction for Graph B is not the fault of the mathematics of the author's method is seen by the fact that whereas his theory calls for 10 per cent of all predictions to be outside the error limits, in Graph B with 212 tests, 9 per cent or 19 tests fall outside, a satisfactory verification of the statistical theory.

The method's weakness lies rather in the fact that it is based on only one variable whereas many variables control the relation between late and early strengths. These variables are not adequately represented in a short series of tests. They include inadvertent changes in fineness of cement, mixing temperatures, and chemical differences in the clinker, each of which can be controlled only within certain practical limits.

For example, the effect of a variation in the chemical composition of the cement of Graph B, Fig. A, in the direction of that used for the job of Graph A, may be seen by glancing at the dashed line C-C connecting the two averages. Under such a condition, an increase in 7-day strength would be reflected by a decrease in 28-day strength. Since the slope of a line expressing 28-7 day relationships can only indicate average results, if chemical variations are present it must be flatter than would be the case for variations produced by changed water-cement ratios alone. In the writer's opinion, no method for forecasting strengths can be expected to be of much practical use unless it includes in its basic data, several independent variables.

For the above reasons, it is thought the author's method of prediction will find its greatest use in laboratory work where unwished for variables may be controlled, or where their effect can be studied by an extension of the author's method involving multiple correlation. Used in this way it is undoubtedly an extremely effective tool. However the author's discussion of "probable error," together with his Table 11, needs some extension before his implication can be accepted that 10 or 12 companion tests are adequate to determine the probable error within a margin of 10 per cent. It is believed that this latter figure, which is determined from a limited number of tests, refers to the fact that on the *average* the computed standard errors are too low and need to be increased by the amounts tabulated in Table 11 if bias is to be avoided.*

Of greater importance than the bias errors of the author's Table 11, is the reproducibility of the "maximum probable error" if computed from

*A more extensive table of correction factors is given in A.S.T.M. Manual on Presentation of Data, Supplement B, p. 50, Table I. The values of C_2 are coefficients by which standard errors computed from a limited test series must be divided to give the unbiased result. Values obtained by this method agree approximately with the author's value.

different test series from the same job. Approximately its per cent error is given by $100 \frac{1}{(2N)^{\frac{1}{2}}}$ where N is the number of tests. For 10 tests its standard error is 22 per cent, or 36 per cent for a 1:10 chance. One would hardly be warranted in changing a mix because a predicted 28-day strength fell below a limit which in itself had 1 chance in 10 of being 36 per cent too high. Only a considerably larger number of tests than 10 will give values of sufficient precision to be used for concrete control.

Finally the writer wishes to raise the question, "Why on a job do we need to make, or to predict the strength of, so many 28-day tests?" Are we not, having made preliminary trial mixes and having set up a control for our materials, as sure of what our strength will be as we are of the lusciousness of mother's pie? Surely we don't need to bite into it to know it is going to be good, provided we have not fallen down in supplying first-class materials. Would not a part of the effort currently spent on compression testing yield more profitable results if it were applied to a more regular and systemmatic control of the uniformity of the manufacturing processes all the way from the cement mill to the finished structure. Lack of control discovered along the assembly line leads to action before damage is done; lack of control discovered 28 days after concrete has grown into a structure only leads to a postmortem and the burial of the corpse. Do not averages of 25 to 40 tests at varying ages up to a year provide all the precision needed for an autopsy, an autopsy principally of benefit to the next job? The writer hopes the author in his closure will comment at least briefly, on these points.

AUTHOR'S CLOSURE

The writer directs attention to a statement on page 494 of the paper: "Parallel specimens must be used in this method. These are defined as concrete specimens of approximately identical mix, type of cement, aggregates, specimen dimensions, and curing."

Mr. Wing does not state, and the writer does not know, whether the specimens represented in Graph B are parallel specimens. From the results, the presumption is that they are not parallel specimens.

Mr. Wing takes the position that variables such as changes in fineness of cement, mixing temperatures and chemical differences in the clinker cannot be represented adequately in a short series of tests. The writer's experience in estimating 28-day strengths over a period of four years, involving hundreds of tests on 3,000-lb standard concretes, and more than 1500 tests of 5,000-lb lightweight concretes, indicates apparently that the above variables can be represented by the condition of parallel specimens.

Mr. Wing believes that the method will find its greatest use in laboratory work, and that, used in this way, it is an extremely effective tool. This conclusion is surprising, inasmuch as all the data presented in the paper, and all the estimates made by the author, have been obtained from job specimens.

Mr. Wing questions the acceptance of ten parallel tests as adequate to determine the probable error with a margin of 10 per cent. He believes that the latter figure refers to the fact that, on the average, the computed standard errors need to be increased by the amounts given in Table 11 if bias is to be avoided. The writer is willing to accept Mr. Wing's opinion, inasmuch as both statements amount to the same thing in practice.

The writer agrees with Mr. Wing that too many 28-day tests are made on a job, and that the 7-day tests should, in general, be sufficient. Unfortunately, the job specifications usually require 28-day tests, and there is very little likelihood that engineers, architects, or specification writers could be persuaded to eliminate the requirement for 28-day tests, unless they were convinced that a reasonably correct picture of 28-day strengths could be obtained by other means. To provide such a method was the principal reason for writing the paper. In addition, a semi-automatic means of evaluating concrete control is thrown in as a by-product.

In closing, the author agrees with Mr. Wing regarding the essential simplicity of the method presented, inasmuch as it can be used with facility by anyone able to read to three places on a slide rule. However, the method is valueless unless the requirement for parallel specimens is strictly observed.

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Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1944 to June 1945.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Concrete in New Caledonia (41-165)

By Capt. K. K. HANSEN*

American soldiers in New Caledonia have found that French colony as backward in matters of paving and plumbing as their fathers found France itself in World War I—and as their brothers are probably finding France today. But in one respect, observant GI's have found this out-of-the-way island amazingly ahead of the United States itself.

Concrete is used in New Caledonia not only for foundations, curbing, culverts, bridges and the like, but also for walls, signposts, fences, telephone and telegraph poles, gates and even village bulletin boards. The only use to which it does not seem to be put is for sewers, which are entirely lacking even in Noumea, the capital city. Noumea normally has a population of 12,000, but has a business district as flourishing as that of an average American city of 50,000, largely because it is the only town of any size on the island, and is the mercantile center for the Loyalty and New Hebrides islands as well as New Caledonia.

The multiplicity of uses to which concrete is put in New Caledonia is the result of preference, and not because there is a lack of other build-

*Courtesy Public Relations Bureau, War Department.

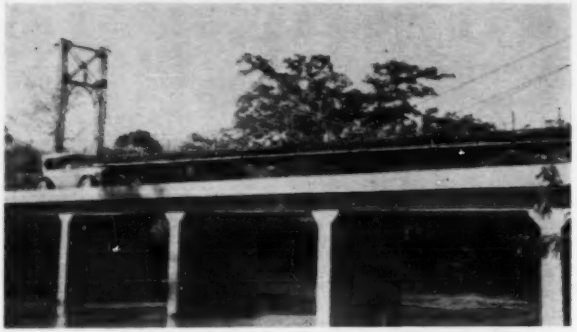

Fig. 1—U. S. Army truck enters La Foa on the new concrete bridge spanning the La Foa River, which, in time of flood, is often higher than the bridge.

Fig. 2—This view of the La Foa River bridge, with an ancient French Renault crossing it, shows the old steel bridge, built in 1908 and now condemned.

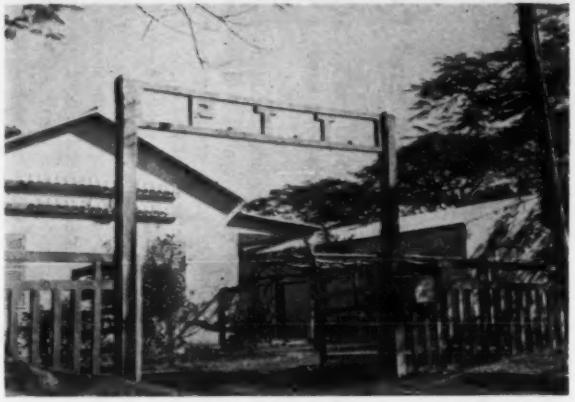

Fig. 3—Post-office gate, sign, fence and the post-office itself in La Foa are all of concrete.

Fig. 4—The side gate to the post-office is also of concrete. "P. T. T." is for "Poste-Telephonie-Telegraphie".

Fig. 5—Signpost on the outskirts of La Foa, a village of perhaps 250 people, 80 miles north of Noumea, is cast in one piece of solid concrete, and stands as high as a man's shoulders. The supports and background for the matrix letters are natural concrete; the letters are painted a light blue and the two horizontal bars dark blue.

Fig. 6—Noumea house with a concrete fence and foundation.

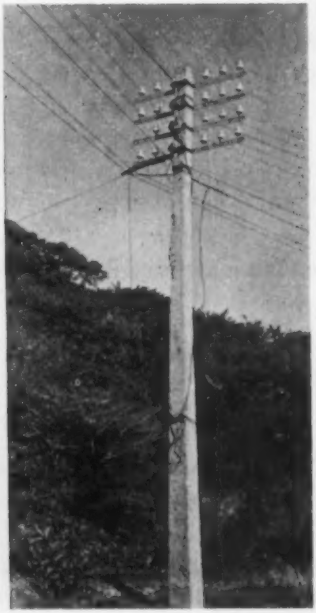

Fig. 7—Even the village bulletin board, in front of the post-office, is of concrete, with a blackboard insert.

Fig. 8—Telephone poles on "U. S. No. 1", New Caledonia are cast in one piece, cross-trees and all, of concrete.

ing materials. The island is about as far south of the equator as the Hawaiian islands are north, and compares to the latter climatically. Many hardwoods are found in the mountains, and the remainder of the island is virtually covered with "niaouli" or paper-bark trees, a species of eucalyptus so resistant to rot and insects—of which there are comparatively few, if one excepts mosquitoes, as compared to other Pacific islands—that it lasts for years when used for fence posts and rails, and even as pilings in salt water, where it will remain sound indefinitely.

The island is rich in iron, chrome and nickel, and there are also a number of deposits of gleaming white sand from which the Melanesian natives make a dazzling white plaster for their wattle houses, which are roofed with paper-bark thatch. The French prefer houses of concrete, with the ubiquitous galvanized-iron roof of the tropics.

Concrete is the favored building material not only in Noumea but in the tiny towns along both coasts of the island. Each village has concrete signposts at its approaches, modernistic in design and built for the ages—signposts which would be a decided improvement on the wooden eyesores which are a blemish on the environs of most American towns.

As one enters the towns themselves, concrete work is in evidence on every hand, some of it, in decorative fences built with concrete on an armature of iron rods, strikingly beautiful. Bridges, too, throughout the island, are of concrete, with no railings because of the torrential tropical rains and the frequent flash floods. The bridges are built so that water can flow over as well as under them, without hindrance.

The accompanying photographs illustrate some examples of New Caledonian artistry in concrete.

Bonding New Topping to Old Concrete Surfaces (41-166)

By F. B. HORNIBROOK* and NATHAN GREEN*

Not infrequently a situation is called to notice where a new mortar or concrete topping had been applied to an old concrete surface and as a result of the failure in bond of the new to the old concrete an unsatisfactory surface exists. Since a new topping undergoes changes in volume as it ages and dries not matched by similar changes in the old concrete to which the topping is applied, strains may be expected to develop at the interface. This, of course, tends to cause separation of the two concretes. A brief series of tests is described, undertaken to ascertain the order of the bond strength between an "old" concrete and a new

*National Bureau of Standards, Washington, D. C.

TABLE 1—TENSILE STRENGTHS DEVELOPED IN BOND OF "NEW" TO "OLD" CONCRETE

(by casting a new topping to a matured concrete surface and after curing of the topping, allowing it to dry to permit drying shrinkage to occur)

Surface Preparation	Bonding Treatment	Tensile strength(1) of bond av. 3 specimens psi	Actual range in measured bond strength psi
Original wood-floated surface	Cement-water slush coat, w/c = 0.5	40	34 to 46
" " " "	1:1 cement to fine sand slush coat, w/c = 0.5	45	38 to 53
" " " "	1:1 cement to powdered iron(2) and rusting agent mix, w/c = 0.5	45	35 to 53
" " " "	Cement-water slush coat plus $\frac{1}{8}$ in. of 1:2 mortar	30	24 to 30
Troweled surface, etched with 1:2 HCl	Cement-water slush coat, w/c = 0.5	35	29 to 43
Chipped to expose coarse aggregate	Cement-water slush coat, w/c = 0.5	95(3)	84 to 111
" " " " "	1:1 cement to fine sand slush coat, w/c = 0.5	95(3)	85 to 103
" " " " "	1:1 cement to powdered iron and rusting agent mix, w/c = 0.5	95(3)	96 to 97
" " " " "	Cement-water slush coat, w/c = 0.5 (topping mixed with 5 gal. sack extra water allowed to stand 1 hr., remixed then applied)	120	119 to 122

(1) Reported to nearest 5 lbs/sq.in.
(2) A proprietary compound recommended by the manufacturer for bonding
(3) Some aggregate pulled out of topping by bond coat

topping after the drying shrinkage of the new topping had been allowed to take place. A few variations in the bonding procedure were also tried to determine the resulting effect on the bond strength.

The test specimens were each prepared as follows: A 6-in. cube was cast and after damp-curing for 7 days was allowed to dry at room temperature for 35 days. This cube represented the "old" concrete. The top face (as cast) was roughened by chipping or acid etching and just prior to application of the new concrete, the roughened surface was soaked with water for 3 hours then air-dried one hour after removal of excess water. The scheduled slush coat was then applied with vigorous scrubbing by means of a stiff-bristled brush, the six-inch cube-mold fitted to the specimen to overlap slightly, and a second cube cast onto this prepared surface. The specimen, consisting of the two joined cubes, was then damp-cured 7 days, air dried 21 days, then tested in tension.

The grips for the tension test were provided by embedding to a depth of about 3 in. in the "rear" of each cube, 8 symmetrically-placed threaded steel rods. The half of each cube containing the threaded bars was filled with a 1:2 mortar, and the "face" half of the first cube (the "old" surface) was filled with a concrete having a nominal cement factor of 6 bags per cu. yd. and ¾ in. maximum size gravel aggregate. The "face" half of the second cube was filled with a topping mix proportioned 1:1 1/3:2 parts by weight of cement, sand, and No. 4 to ⅜-in. pea gravel, with a nominal water content of 4½ gal. per bag. This topping is patterned after the recommendations of the Committee 804 report, "Suggested Recommended Practice for Wearing Surfaces for Floors" (A.C.I. Jl. Sept., 1938; *Proceedings* V. 35, p. 21).

Table 1 lists the various bonding procedures used and the tensile strengths attained. In all cases the break occurred in the bond of the "new" to the "old".

From the results in this table it is evident that under the conditions of this test even the most favorable bonding procedure failed by a wide margin of developing strengths approximating the normal tensile strength of concrete. Considering the various bonding procedures it is indicated that (1) a thorough chipping and roughening of the old surface was essential; (2) none of the slush coats tried was significantly more effective than the plain cement-water application; and (3) that pre-aging the topping mix for an hour then remixing improved the bond strength materially. This latter procedure, while probably impractical on a large job, should be well worth while on small patching jobs.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Decrease of the detrimental influence of organic impurities in concrete sand

INGE LYSE, Kgl Norske Videnskab. *Selskab Forh.* 16, 43-6 (1943); *Chem. Zentr.* 1943, II, 2197. *Chemical Abstracts*, V. 38, No. 21, Nov. 10, 1944. HIGHWAY RESEARCH ABSTRACTS

By speeding up the binding and hardening process the detrimental influence of organic impurities can be decreased. The speeding up is accomplished by use of $CaCl_2$, $MgCl_2$, $AlCl_2$, and $FeCl_2$. $CaCl_2$ proved to be best.

Bridge Foundations

W. A. FAIRHURST, *Concrete and Constructional Engineering*, V. 39, p. 304-307
Reviewed by GLENN MURPHY

This installment includes several drawings for the Inverbervie bridge which is a multiple span girder bridge on a curve of about 420 ft. radius, and the bridge at Guardbridge, also a multiple span girder. Details of foundations, piers, hinges, and rockers are shown.

Flexural strength of concrete

STANTON WALKER, before annual meeting of Board of Directors of Nationl Sand & Gravel Assn., Jan., 1945 —mimeographed.

Discusses in elementary terms significance of flexural strength of concrete, particularly in relation to road slabs. Reviews data from various sources on the several factors affecting flexural strength, including angularity of aggregates, surface texture of aggregates, strength of coarse aggregate, etc. Suggests that flexural strength is better measure of deterioration of concrete than compressive strength.

Bridge foundations

W. A. FAIRHURST, *Concrete and Constructional Engineering*, V. 39, No. 11 (Nov., 1944), pp. 277-279
Reviewed by GLENN MURPHY

The author continues his series with photographs, drawings and brief descriptions of three arch bridges. The Glen Bridge, Dumfermline has a span of nearly 190 ft. for a 28-ft. roadway with 6-ft. walks on each side. The Kemnay Bridge, Aberdeenshire, carries a 19-ft. roadway ad has a clear span of 122 ft. The Linlithgow Bridge has a clear span of 108 ft. and an overall width of 53 ft. Details of foundations are shown for each of the structures.

Lightweight concrete aggregates

Concrete and Constructional Engineering, V. 39, No. 11 (Nov., 1944), pp. 289-290

Reviewed by GLENN MURPHY

A new edition of Building Research Bulletin No. 15 "Lightweight Concrete Aggregates" by Dr. F. M. Lea is available from H. H. Stationery Office, London (Price 3 d.). Data are given comparing the properties of concretes made with coke breeze, pumice, foamed slag and expanded slate aggregates. Some values of compressive strength, unit weight, shrinkage, expansion, and thermal conductivity are abstracted in this article.

Concrete caissons at the Normandy harbor

Concrete and Constructional Engineering, V. 39, No. 11 (Nov., 1944), pp. 280-283

Reviewed by GLENN MURPHY

This is a general descriptive article concerning the 146 caissons which were constructed in England and towed across the channel to provide two artificial harbours each approximately the size of Dover Harbour for the landing of troops and equipment on a large scale. The caissons were constructed in six sizes ranging from 1672 to 6044 tons displacement. Their construction required approximately 330,000 cu. yd. of concrete, 31000 tons of steel, and 1500000 yd. of forms.

Experimental houses built with lightweight concrete

Concrete and Constructional Engineering, V. 39, No. 11 (Nov., 1944), pp. 284-285

Reviewed by GLENN MURPHY

Of several experimental houses built at Northolt by the Ministry of Works two pair are of steel frame construction with concrete slab filling, one pair is "no-fines" concrete, one pair is concrete with foamed slag aggregate, and one pair is concrete with expanded clay aggregate. These houses, constructed to provide information for postwar housing schemes, are to be compared with houses with 11-in. cavity brick walls. All houses have a floor area of 850 sq. ft. Details of construction and mixtures are given. One feature was the use of expanded metal (with a diamond mesh ½-in. by ¼-in.) for form The mesh retained the concrete satisfactorily and the rough texture provided an excellent key for plastering and external rendering.

Construction of a bridge over a railway

L. E. HUNTER, *Concrete and Constructional Engineering*, V. 39, No. 7 (July, 1944), pp. 177-181

Reviewed by GLENN MURPHY

This article describes some of the construction details involved in the erection of a bridge over a railway and highway. The 18-in. road slab of the bridge is supported by steel joists with a clear span of 26 ft. 4 in. over the railroad and a clear span of over 15 ft. 6 in. over the highway. The joists are supported by 3 ft. piers at the ends and a 4 ft. thick intermediate pier between the railroad and highway. The piers are supported on bearing slabs approximately 2 ft. 6 in. thick. Footings for the bearing slabs were obtained by driving sheet piling through the underlying soil and peat to a sand bottom about 15 ft. below the surface. A description of the system of forms used in the concreting is given in detail.

Concrete surface finishes, renderings and terrazo

W. S. GRAY and H. L. CHILDE, London, 1944 (Concrete Publications, Ltd.), 9½ in. by 6½ in., pp. 137, figs. 132, 8*s.* 6*d.*

The book is a practical treatise on present methods of carrying out surface finish treatments for concrete whereby both color and texture are improved. Many photo-

graphs serve to illustrate modern trends in the treatment of concrete surfaces including the development of "architectural concrete" through careful attention to design and surface finish. Chapter headings are as follows:—Causes and prevention of blemishes. Cement washing and rubbing down. Concrete in architecture. Pre-cast slabs as shuttering. Sliding shutters for special concrete faces. Exposed aggregate surfaces. Paints, stains, and gunite. Pre-cast units. Renderings and kindred surfaces. Interior finishes. Terrazzo and mosaic. Artificial marble.

Shear reinforcement in column bases

W. T. MARSHALL, *Concrete and Constructional Engineering*, V. 39, No. 9, Sept., 1944, pp. 219-223
Reviewed by GLENN MURPHY

The author presents a method of design of reinforcement for column footings based on the assumption that the critical section in shear is on a surface extending downward from the base of the column at an angle of 45 deg. with the horizontal. The assumption is based on the results of tests conducted by the author and published in the *Journal of Inst. C. E.*, March, 1944. The proposed design involves two sets of reinforcing bars: one set being the conventional tensile reinforcement in the bottom of the footing; the other, a set along the bottom of the fotting under the column then extending upward and outward at 45 deg. to the top of the footing, and then along the top of the footing to the outside. Hooks are provided on each set. Design calculations are given for two examples.

Comparison of standard and proposed tension grips and test pieces for testing portland cement mortars

MATTHEW MCNEARY, Bulletin 39, Maine Technology Experiment Station, University of Maine, April, 1944.
HIGHWAY RESEARCH ABSTRACTS

The standard tension briquet, adopted by the American Society for Testing Materials, does not give a true measurement of the tensile strength of portland cement mortar because of (1) stress concentrations at the grips, (2) higher stresses at the outside of the neck than at the center, and (3) cross stresses at the neck, which obviate a condition of pure tension at the break. Photo-elastic studies have shown that all three of these conditions could be improved by using rubber grips instead of the metal roller type and by elongating the neck of the briquet by inserting a straight section one inch long.

Several hundred briquets of both the standard and elongated type were made and tested in both the standard grips and the rubber grips with the following results: 1. The changed briquet shape and the rubber grips each separately caused an appreciable increase in the apparent tensile strength of the mortar. 2. The rubber grips proved to have the important effect of narrowing the range of variation of the strength of three briquets taken from the same three-gang mold.

Overlapping tensile reinforcing rods in concrete beams

R. H. EVANS, *Concrete and Constructional Engineering*, V. 39, No. 7 (July, 1944), pp. 167-175
Reviewed by GLENN MURPHY

This paper presents the test results obtained from a series of concrete beams in which the reinforcing steel consisted of hooked bars 2 or 3 ft. longer than half the span. The bars were lapped (a) where the bending moment was constant, (b) where the shear was constant.

Fifteen beams were tested, each having a cross section 5 in. by 10 in. and a span of 100 in. with loads at the third points. A 1:2:4 mixture with high alumina cement was used for all beams. Various arrangements and sizes of high strength steel reinforcing bars were used. Strains along the steel were measured with a 4-inch strain gage in

several of the beams immediately after the beam cracked in tension and again just before final failure.

The results show that simple overlapping of the reinforcing rods does not form an effective joint. At low loads, cracks appear at the ends of the overlaps and develop into diagonal tension cracks, resulting in collapse of the beam at loads well below those carried by a beam with continuous reinforcement. The recommendation is made that if full length rods are not available the shorter lengths welded together rather than simply overlapped.

Post-war building studies No. 8—Reinforced concrete structures

MINISTRY OF WORKS, London, 1944 (H. M. Stationery Office), 9½ in. by 6 in., pp. 12, 6*d*.

The Reinforced Concrete Structures Committee which was convened by the Institution of Structural Engineers in September, 1942, here presents its recommendations for ensuring in the postwar period the maximum economy of material and the most rapid methods of reinforced concrete construction having due regard to other interrelated building services such as lighting, ventilation, plumbing, etc. The report comprises the following sections:—1. Consideration of Code of Practice; a new national Code of Practice for reinforced concrete, containing a power of waiver and to be revised every 3 years, is recommended. 2. Loads on floors and roofs. 3. Stresses in steel and concrete. The stresses in concrete in compression due to bending may be raised by 10 per cent above those given in the code issued in 1939 by British Industries National Council (see *B.S.A.*, 1940, No. 660). 4. Improvement in design and construction methods. Emphasis is laid on the employment of only properly qualified and trained persons for the design and construction of reinforced concrete. Suggestions for effecting economy of time are also enumerated. 5. Loan periods. 6. Reinforced concrete foundations. 7. Prestressed and vibrated concrete. 8. Welding; it is recommended that, at present only, butt welding be allowed for mild steel. 9. Composite construction. 10. Code of Practice: certain modifications to the existing B.I.N.C. Code are recommended and enumerated. The two appendices list the Institution of Structural Engineers' schedule of symbols recommended for use in reinforced concrete and structural steelwork calculations, respectively.

Expansion of clay and concrete drain tile due to increase of temperature and moisture content

DALTON G. MILLER and CHARLES G. SNYDER, *Agricultural Engineering*, May, 1944, V. 25, No. 5. (Abstracted by M. S. Kersten). HIGHWAY RESEARCH ABSTRACTS

Comparisons of the amount of expansion in clay and concrete drain tile due to temperature increase and change of moisture content is provided by the results of tests on products from 16 plants in Minnesota and Iowa.

The average coefficient of thermal expansion for the clay tile pipe was 0.0000025 per degree Fahrenheit. The average expansion due to wetting from a room dry condition to 28 days submerged was 0.00005-ft. per ft. of length; the average absorption in this time was 9.7 per cent. Thus the expansion due to each per cent of moisture absorbed is equivalent to that of a 2-deg. F. rise in temperature.

For the concrete pipe the average thermal coefficient was 0.0000030 per deg. F. and the average expansion due to a 10.1 per cent absorption was 0.00046 ft. per ft. A moisture increase of 1 per cent gives an expansion equivalent to that caused by a 15-deg. F., rise in temperature.

The authors conclude that expansion of clay tile after installation due either to increase of temperature or to increase in moisture content is not of sufficient magnitude to impair

the effectiveness of the tile line through closure of the joints between individual tile. The same may be said of dry-tamped concrete tile as regards expansion due to temperature changes, but some precaution should be exercised not to install concrete tile when too dry.

Shape, size, and shrinkage

A. D. Ross, *Concrete and Constructional Engineering*, V. 39, No. 8 (Aug., 1944), pp. 193-199
Reviewed by GLENN MURPHY

The objectives of the investigation reported were to (a) determine qualitatively the shrinkage in specimens of widely differing dimensions and shape, (b) correlate shrinkage and loss of weight by drying, (c) investigate differential shrinkage, (d) test the applicability of the diffusion and surface emission equations to mortar drying as a porous solid, and (e) determine the magnitude of autogenous shrinkage. Specimens were made of a 1:2 standard Portland cement mortar with a water-cement ratio of 0.40 by weight, and were stored in a room maintained at a mean temperature of 71 F. and a mean relative humidity of 53 per cent. Linear measurements were taken between steel balls cast in the specimens, using a frame with a built-in micrometer. Autogenous shrinkage was measured by displacement, immersing the specimens in mercury. Specimens were 9 in. long with rectangular, square, triangular, circular, and annular cross-sections. From two to four sizes of each shape were used to obtain different values of the surface-volume ratio. Shrinkage and loss of weight curves for times up to 160 days are shown.

The conclusions were that size and shape have a pronounced effect upon the magnitude of the shrinkage, and that the variation is largely a function of the surface-volume ratio The rate of shrinkage is not a fixed property of the material, but may be evaluated in terms of the surface-volume ratio. Shrinkage extends only a short distance into the interior of large masses.

It is unfortunate that destruction of equipment and specimens by enemy action prevented completion of the project.

1944 book of A.S.T.M. standards

In three parts (1235 specifications and tests, 6030 pages, American Society for Testing Materials ($30.00 or $10.00 per part.)

The new 1944 Book of A.S.T.M. Standards, issued in three parts contains in latest approved form all of the society's widely used specifications and tests for materials—1235 specifications and standard methods which cover more than 6000 pages.

All specifications, whether formal standards or tentative are given. The three parts, are: *Part I, Metals.*—Ferrous and non-ferrous metals (all A and B and some E serial designations) except methods of chemical analysis. General testing methods (E serial designations;) *Part II, Nonmetallic Materials*—Constructional.—Cementitious materials, concrete and aggregates, masonry building units, ceramics, pipe and tile, thermal insulating materials (all C serial designations); wood and wood preservatives, paints, varnishes and lacquers, road materials, waterproofing and roofing materials, soils (certain D serial designations). General testing methods, thermometers (E serial designations); *Part III, Nonmetallic Materials—General.*—Fuels, petroleum products, electrical insulating materials, rubber, textiles, soaps and detergents, paper, plastics, water (remainder of D serial designations). General testing methods, thermometers (E serial designations).

This 1944 Book includes all emergency standards and emergency alternate provisions which have been widely used to expedite production and procurement of important materials (a separate volume, Chemical Analysis of Metals, includes standards and

recommendations for both ferrous and non-ferrous metals. These methods are not included in the Book of Standards.) Each part of the 1944 Book has a complete subject index (from 36 to 44 pages).

To keep the books up to date, a supplement will be issued to *each* part late in 1945. As a service with the 1944 Book of Standards there is a complete 200 page Index to Standards, which is furnished without additional charge and a copy accompanies the purchase of each part or complete set. (1945 Supplement will cost $4.00 per part).

Effect of tar, ammonium fluosilicate, and sodium hydroxide on the alkali resistance of concrete

E. C. E. Lord, *Pub. Roads*, 1944, 23 (11), 282-96.

Exposure tests have been made on the protective value of water-gas tar, ammonium fluosilicate and sodium hydroxide, either alone or in combination, for concrete exposed to the action of sulphate waters. The tests were made on concrete specimens (3-in. by 6-in. cylinders) having water/cement ratios of 0.7 and 0.8 by volume and using two different cements, one having a low alumina-iron ratio and the other a high alumina-iron ratio. The specimens were given an intial curing in the moist room of 7 days and 28 days, followed by curing in laboratory air for periods of 1 and 7 days for those specimens to be treated with tar, and 7 and 28 days for those specimens to be treated with ammonium fluosilicate and sodium hydroxide. The protective materials were then applied by immersion and by brush coating followed in some cases by a seal coat of coal tar applied with a brush. The treated specimens, when dry, were stored, some in 5 per cent sulphate solution in the laboratory, and some in sulphate waters (2.4 to 2.7 per cent sulphates) of Medicine Lake, South Dakota, for periods of several years. The results of these exposure tests are shown in photographs and tables and are discussed. The trend of performance of the specimens stored in the laboratory solutions and of those stored in Medicine Lake were in comparatively close agreement. Concrete made with the low alumina/iron ratio was more resistant to sulphate action than that made with the high alumina/iron ratio. Water-gas tar applied either by immersion or by brush gave the same degree of protection which was greatly increased by the addition of a seal coat of coal tar. Immersion beyond 1 hour or the application of more than 4 brush coats produced no appreciable benefit. Ammonium fluosilicate, either alone or combined with tar, did not give adequate protection; sodium hydroxide, either alone or combined with tar, gave somewhat better protection but was still distinctly inferior to the tar treatment alone. Character and curing of the concrete influenced the resistance to sulphate waters; concrete made with low water/cement ratio was more resistant than concrete with a high water/cement ratio. Concrete cured 1 day in air prior to treatment was more resistant than concrete cured 7 days.

Precast concrete bridge

L. J. Eichelgrun, *Concrete and Constructional Engineering*, V. 39, p. 293-303

Reviewed by Glenn Murphy

Several general applications of precast concrete to bridge construction are described. Most of the structures discussed in the article were built or designed for the Great Western Railroad. The first application is the use of precast concrete slabs for replacing decks on bridges. In the example described, standard slabs 8 inches thick and 3 feet wide were used for a 60-ft., single track railroad bridge deck. The slabs were laid on a hardwood backing placed between the protruding unit heads of the girders as a cement mortar backing was found to be unsatisfactory. The slabs were necessarily heavily reinforced.

The second application makes use of precast girders in new construction. In one example listed U-shaped girders are used. The girders are laid side by side giving a

beam-and-slab effect in cross section. As the girders act independently, a thick layer of ballast is recommended to help distribute the wheel load. Another example utilizes the standard girder-and-slab construction. In a third example the slab was cast integral with the two main girders and the unit lifted into place after the forms had been stripped. The bridge described had a length of 47 feet and the slab and girder unit weighed about 16 tons.

Precast concrete units have been used for foot bridges. In one type of construction a T-beam is used, the top of the T forming the 5-foot footway. The parapets are attached to the sides of the T-beam. In another type of construction narrow beams about 6 feet deep run longitudinally on each side, the upper part of the beams forming the parapet. The walkway consists of a 5-foot slab expanding between the beams. The slab rests on shoulders 8 inches above the bottom of the beams.

Precast U-shaped sections have been used as launders (flumes). In the example shown, the sections have an overall height of 5 ft. 7 in., a width of 4 ft. 3 in. and a length of 21 ft. 6 in. The walls are 6 inches thick and the base 5 inches thick. The sections were waterproofed with two coats of tar.

The use of precast slabs for forms is mentioned and several design and construction hints are given. Drawings and photographs are included.

Rate of sedimentation

Ind. Eng. Chem. 36, 618-624, 840-847, 901-907 (1944);
(Bulletin 3, Portland Cement Assn Research Laboratory)

Three papers report the results of a study of the fundamentals of sedimentation in thick suspensions such as portland cement pastes. This study was a direct outgrowth of the work reported in 1939 by Powers on the bleeding of portland cement paste, mortar and concrete.* In that study the bleeding of paste and concrete was shown to be a special case of sedimentation, and equations were developed showing the role played by water content and surface area. The equations contained an empirical constant for which only a tentative interpretation could be offered. When in 1940 Dr. Steinour took over the work, his aim was to determine whether the empirical constant merely serves to compensate for fundamental misconceptions embodied in the equation or whether it represents physical characteristics of the suspension and if so to determine what the characteristics are.

Dr. Steinour confirmed the fundamental concepts of the original work but showed the need for modifying some of the mathematical expressions. However, Dr. Steinour's papers go much beyond the matter of confirming earlier work. They provide a general understanding of the behavior of thick suspensions that should be valuable wherever such suspensions are encountered.

The first of the three papers deals with non-flocculated suspensions of spheres; the second deals with uniform-size angular particles in both the flocculated and non-flocculated states. Comparisons of the behaviors of these suspensions with those of non-flocculated spheres showed the effects of particle angularity and flocculation. The third paper takes up sedimentation of various pulverized minerals embracing a wide range of particle sizes. In general the sedimentation of these mineral powders was found to follow the same laws as the simplified systems dealt with in the first two papers.

To the investigator of problems in cement and concrete, these papers not only round out information on the phenomenon of bleeding, but they give also insight into the structure of fresh cement pastes. For example, the studies indicate that a normal portland cement paste is not composed of individual clusters of cement grains; rather, the particles are linked together into one continuous "floc" in which the individual particles are evidently held in place by interparticle forces.

*J. Am. Concrete Inst., June 1939; *Proceedings* V. 35, p. 465 (Bulletin 2, PCA Research Laboratory June 1939).

Contribution to the question of the lime-solubility of portland cement and mixed cement on a portland cement basis

R. ZOLLINGER, *Zement*, 1943, 32 (17/18), 187-96.

The investigation described was carried out in connection with the selection of a cement for concrete exposed to the action of water containing free carbon dioxide. The 12 test cements included portland cement, "Erz" cement, iron-portland and portland-blastfurnace cements, also trass cement, containing portland cement clinker: trass in the proportions 70:30 or 60:40, and cement with added "Thurament," and artificial trass. The problem to be studied was the extent to which the protective influence of the reactive silica added to the cement results from chemical reaction as distinct from the physical effect of increased density due to the colloidal volume increase. The latter effect was excluded by the use of porous test specimens into the interior of which the water could penetrate. The cement-sand (1:5.5) specimens were prepared in the form of slabs, 20 cm. by 20 cm. by 2 cm., and were exposed in separate glass tanks to a continuous flow of water containing carbon dioxide which was changed when the lime content exceeded 0.2 g./l. At intervals during a period of 210 days the quantity of lime dissolved from the slabs was determined from analysis of the tank water. On conclusion of the experiments the specimens were removed from the water and, the disintegrated material having been brushed off, were stored in damp air for 7 days. The specimens were then weighed and the loss of weight obtained, the original weight being known. The loss of weight of the residual slabs exceeded the weight of the removed, disintegrated material; the amount of the additional loss of weight was an indication of the extent to which the slabs had been attacked. The data of loss of weight and lime dissolved and the lime solubility curves of the test cements are shown. The tensile strength of the test slabs at the end of the test period of 234 days and of other slabs at the end of the same period was determined by bending strength tests. The strength test results showed not only a redction of strength in each case but also a complete rearrangement of the strength values of the different cements. A second series of experiments was carried out with slabs of the same cements at the end of a storage period of 6 months in damp air to allow time for a chemical reaction similar in nature to the protective reaction to take place. The results obtained with each test series are considered and the test cements placed in order according to the degree of corrosion, lime solubility and the effect on tensile strength. A decisive statement respecting the value of the individual cements, on the basis of the experimental data, it is pointed out, would be misleading. It may be said definitely, however, that in no case is the chemical protective action adequate to prevent an attack by corrosive waters. Chemical reactions with protective effect undoubtedly take place. The age of the concrete is not without importance, but complete protection is not attained with increasing age and different types of cement differ considerably in their behaviour. Where, in practice, resistance to the action of corrosive water is observed it is due in part to the physical, waterproofing effect of the colloidal compounds.

Highway Research Board—synopses of papers, 24th Annual Proceedings

HIGHWAY RESEARCH BOARD ABSTRACTS

The 24th Annual Meeting of the Highway Research Board, scheduled for Cincinnati Ohio on Nov. 22-25, 1944, was first postponed—later cancelled.

The major part of the value of the year's research work will be preserved by publication in Vol. 24 of H. R. B. Proceedings.

A special issue of Highway Research Abstracts contains summaries of practically all of the papers presented for publication. Synopses of those of major interest in relation to concrete, follow:

Structural efficiency of transverse weakened-plane joints

E. C. SUTHERLAND, Senior Highway Engineer and H. D. CASHELL, Associate Highway Engineer, Public Roads Administration (reported by Highway Research Board, Committee on Rigid Pavement Design, R. D. BRADBURY, Chairman).

The investigation was made as a cooperative research between the States of California, Kentucky, Michigan, Minnesota, Missouri and Oregon and the Public Roads Administration. In each State an experimental pavement several miles in length, embodying the experimental features has been constructed and kept continuously under observation. All of the projects have been described before the Board with the exception of that in California (see Proceedings of the 20th and 21st Annual Meetings). In addition to these experimental pavements in service in the several States the original program included a study of the structural efficiency of transverse joints of the weakened-plane type to be made by the Public Roads Administration. The first description of this work is in the current symposium.

Briefly, the experimental features common to the six State projects consist of a series of plain and reinforced concrete sections in which the expansion joint spacing is varied from 120 to 5,280 feet. All of the plain concrete sections have transverse contraction joints at relatively close spacing (15 to 25 feet) while the reinforced sections have expansion joints at 120-foot spacing with one intermediate contraction joint.

In general, load transfer devices were used in all expansion joints but were used in only part of the contraction joints of a given project in order to determine whether or not load transfer is needed with closely spaced contraction joints of the weakened-plane type. Several of the States included in their projects additional experimental features of design that were of particular interest to them. These features are described in the reports published.

During the three or, in three cases, four years since these pavements were constructed measurements and observations have been made of the following:

1. Daily and seasonal variations in temperature.

2. Daily, seasonal and progressive or permanent changes in the widths of the expansion and contraction joints.

3. Changes in elevation of the pavement, especially with respect to faulting at the joints.

4. The general condition of the pavement and joints.

Progress reports describing the condition of the pavements in the respective States and presenting the data collected up to this time have been prepared by each of the States participating in this investigation and are being presented in this symposium. In addition, the California project is described for the first time.

The authors of these reports feel that it is too early to draw final conclusions regarding the merits of the different designs, but they have made some interesting observations with respect to the data and the significance of certain trends indicated by the data. Some of the tentative indications that appear in the present reports are:

1. Where expansion joint fillers of a plastic type are used there has been a progressive closing of the expansion joints and a progressive opening of the contraction joints with time. The greatest progressive change in the width of the joints occurs during the first year after the pavement is laid and it appears to continue so long as there is expansion space available. The magnitude of the progressive opening at the contraction joints appears to be related to the spacing of the expansion joints, being less for the greater expansion joint spacings.

2. Wood expansion joint filler as used in one project appears to restrain the progressive changes in joint width.

3. The seasonal changes in width of the contraction joints seem to increase as the spacing between these joints is increased, but this variation may not be linear if the expansion of the pavement is restrained during periods of high temperature.

4. In pavements laid at temperatures less than those which normally occur in the summer, the seasonal changes in the widths of the contraction joints seem to be less in pavements with limited expansion space than in pavements with greater expansion space.

5. While a limited amount of faulting has been noted in some of the pavements, the amount is not sufficient to justify any conclusions regarding the necessity for the use of dowels to prevent faulting in weakened-plane joints.

As an integral part of the general investigation the effectiveness of aggregate interlock for reducing the critical stresses caused by loads, acting in the vicinity of weakened-plane joints, has been studied by the Public Roads Administration on a test pavement near Washington, D. C. This pavement consisted of six 30- by 20-foot sections of 8-inch uniform thicknesses, each divided longitudinally by a deformed metal plate center joint and transversely by a weakened-plane joint. The type and maximum size of coarse aggregate in the concrete was varied in the different sections so that the effect of these variables on the efficiency of weakened-plane joints might be studied.

In making the tests to determine the efficiency of the joints, loads were applied at the joint edges, free edges and at the interior of the slabs and the critical strains caused by these loads were measured. Tests at the joint edges were made with the joints in a closed condition (under compression) and at various controlled openings.

It was found that all of the weakened-plane joints were effective in reducing the critical load stresses when closed and under compression, but that aggregate interlock was not a dependable method of stress reduction when the joints were open small amounts.

Structural Behavior of Concrete Airfield Pavements, R. R. Philippe, U.S. Engineer Department, Cincinnati Testing Laboratory. The years immediately before the war and those since its beginning have witnessed a tremendous development of all types of military aircraft. Not the least spectacular of these developments is the advancement of the heavy land-based bomber and cargo ship. Today we read of building pavements for 150 kip wheel loads, a six-fold rise in as many years from a wheel load standpoint. Experts in aircraft design speculate sagely that it would not be safe to guess that heavy aircraft of the future will weigh less than a million pounds.

These heavy aircraft, when used as bombers, must possess long range and large bomb carrying capacity. This demand, coupled with the structural difficulties of multiple landing gear, has resulted in a marked tendency on the part of plane designers to concentrate severely the loads of planes on the minimum number of wheels, thereby multiplying the need for heavier and heavier pavements.

The Corps of Engineers has met the challenge of these designs by accelerated testing of extrapolated designs. The investigations for rigid pavements have been centered largely in the Ohio River Division, where its unit, the Cincinnati Testing Laboratory, has been designated as the Engineer Department's Rigid Pavement Laboratory.

The tests, to date, have been designed to determine the effect of impact of a landing wheel of variable load on a rigid pavement, to measure the reactions of a pavement under a set of idealized conditions as assumed by Westergaard's theory, and to study the effect of accelerated traffic on rigid pavements. Impact effects were studied by

means of flight tests and drop tests, wherein the reactions both in the plane and in the pavement were correlated. Static loading tests were employed to check the validity of theoretical assumptions, and later to correlate the effects of repeated loading to the results observed in traffic tests. Accelerated traffic tests with wheel loads ranging from 20 to 60 kip have been conducted on thirty-seven designs, and all preparations have been made to traffic test nineteen basic designs with a 150 kip wheel load.

The paper is a description of the methods employed in testing and presenting the factual data of the designs tested. It establishes for record the basis for future presentation and discussion of results. A few of the more startling results are illustrated to stimulate interest by the reader.

Progress Report of the Committee on Durability of Concrete, M. O. Withey, Chairman, University of Wisconsin. This committee, an outgrowth of the Project Committee on Durability of Concrete as Affected by the Cement, was formed early in 1940. The object in its formation was to consider and investigate factors related to the durability of concrete. A reasonably rapid method for ascertaining the resistance of concrete to freezing and thawing has long been desired by materials testing engineers, but to date such procedures as have been stipulated in specifications have failed to obtain a large following.

Since the Project Committee on Durability of Concrete as Affected by the Cement had made some tests in which the effects of different rates of freezing on resistance of mortars to freezing and thawing had been observed, it seemed desirable to give further study to this important subject. In arriving at this decision the committee was well aware that the results of further fundamental research on the causes of deterioration due to freezing and thawing might materially modify such testing procedures as the committee might now use or prescribe. Nevertheless the committee felt that the uncertainty of the time at which such fundamental information would be available and the immediate need of a suitable testing procedure were ample justification for the prosecution of the proposed program.

Program.—Essentially the program adopted involved: (1) a comparison of the relative severity of a carefully specified coordinating freezing and thawing test as practiced in different laboratories. (2) a comparison of the effects of the freezing and thawing procedures commonly used in these laboratories (local procedures). (3) a comparison of the severity of the coordinating test procedure with the local laboratory procedures.

Tests providing these comparisons were made in seven laboratories, and this report describes the testing procedures, discusses the results and contains the following conclusions: 1. Under the conditions of these tests the electronic vibrating devices used provided a convenient and rapid means of determining the change in the dynamic modulus of elasticity of the specimens tested. 2. Within the limits of these tests the average relation of the percentage decrease in modulus of rupture (R) to the percentage decrease in the dynamic modulus of elasticity (E) due to freezing and thawing was $R = 1.5E$ for either the local or coordinating test procedures. Considering the data of laboratories performing the entire B and C test programs the individual laboratory average R — E relations were within 20 per cent of the above grand average. 3. The relation between the percentage increase in the modulus of rupture (R′) and the percentage increase in dynamic modulus of elasticity (E′) of the moist-cured control beams was much more variable than the relation between the decreases of these properties in freezing and thawing. Based on changes after 28-days the average relation was $R' = 1.2E'$. 4. For the types of concrete tested the relation between the reductions in flexural strength and in dynamic modulus of elasticity was sufficiently reliable to measure the

rate of deterioration of the flexural strength under the methods of freezing and thawing used. 5. The flexural strength is much more sensitive to the deteriorating effects of freezing and thawing than is the compressive strength. 6. Within the limitations of these tests the relation of the reduction in the compressive strength to the reduction in the dynamic modulus of elasticity due to freezing and thawing was so variable in the tests conducted that the reduction in the dynamic modulus could not be used as a measure of the reduction in compressive strength. 7. Loss in weight does not provide a criterion of the early deterioration in flexural strength due to freezing and thawing. 8. Specimens frozen in air as in the coordinating program showed little surface deterioration due to breakdown of the mortar. Some specimens exhibited spalling over unsound coarse aggregate particles. 9. Specimens frozen in contact with water evinced deterioration at the corners and edges and over the portions partially immersed in water. 10. Although there are exceptions, a comparison of the local test procedures on the basis of the B and C specimens indicates that in general those procedures in which the rates of freezing from 32 to 15 F. were fast caused failure more quickly than those in which the rates were slow. 11. Those local test procedures having fastest rates of freezing and producing quickest failures did not discriminate clearly between the concretes made of satisfactory and those made of poor coarse aggregate, whereas the procedures in which the rates were somewhat slower and the number of cycles to failure greater provided good discrimination. 12. None of the freezing and thawing procedures tried provided a small dispersion in the number of cycles required for failure and a sufficiently high degree of discrimination to qualify as a standard method. Of the procedures used, the coordinating program, the local Wisconsin Highway Commission, and the Missouri Highway Department are the best, but all exhibit too great dispersions of individual test values to be considered satisfactory. With still better control of the variables it is believed that these dispersions can be reduced and a standard procedure established. 13. The accumulated data emphasize the necessity for regulating carefully the methods of making and curing specimens, the air content of the specimens, the degree of saturation of the aggregate at the time of making, and the degree of saturation of the concrete at the time of freezing. 14. Curing of concrete in 70-deg. water subsequent to deteriorating freezing and thawing treatments produces a marked recovery in the dynamic modulus of elasticity but a much less pronounced recovery in flexural strength. 15. In these tests neither the absorption nor the rate of absorption data for the concrete correlated with the differences which the beams exhibited in resistance to freezing and thawing.

Thermosetting Synthetic Resin Paints for Concrete Pavement Markings, Floyd O. Slate Research Engineer, Joint Highway Research Project, Purdue University. The purpose of this study was to find the causes of failure of present concrete highway paints, and to find new or different paints better able to withstand these causes of failure.

The effect of passage of water vapor through a paint film on concrete was studied by means of evaporation tests. It was found that the passage of water upward upon evaporation caused the deposit of salt crystals at the concrete surface. The effect of this crystal growth on paint films was studied. Photographic records are reproduced in part. Paint lines put down in the field were observed carefully for type and cause of failure.

Synthetic resin paints, chosen as likely to be superior to standard highway paints, were subjected to water, alkali, and abrasion resistance tests. Field and laboratory comparisons were made with the standard paints. Thermosetting synthetic resin paints were baked directly on concrete pavement by means of infra-red lamps.

Influence of Various Curing Methods on some Physical Properties of Portland Cement Concrete, H. C. Vollmer, Research Associate, National Bureau of Standards. An in-

vestigation has been undertaken to compare the effectiveness of various curing materials and procedures. This investigation includes the use of burlap, the use of several liquid curing compounds and the use of calcium chloride both integrally and as a surface application. The study includes tests of concretes cast and cured at 70 F and at a relative humidity of 50-60 per cent, concretes cast and cured at 100 F. and at a relative humidity of 25-35 per cent, and concretes cast and cured under field conditions. The evaluation tests include flexural strengths of beams, compressive strengths of beam ends and resistance to abrasion of the cured surfaces. This report, presented as a progress report, presents only results of tests on the concretes cast and cured at 70 F.

The use of damp burlap for 18 hours and with calcium chloride used either integrally in the concrete or spread on the surface of the concrete upon removal of the burlap, resulted in 28 day flexural strengths of the same order as obtained with wet burlap applied for 3 days, the accepted standard for highway construction. The use of surface calcium chloride applied as soon as the bleeding water disappeared (no burlap at all) resulted in strengths only some 7 per cent lower than those obtained by the procedure requiring burlap. The 28 day flexural strengths obtained by the use of liquid curing compounds were 16 to 19 per cent lower than those obtained with the 3 day burlap curing procedure. Flexural strengths obtained by continuous damp curing of the specimens for 28 days, which however is not a practical method under field conditions, were higher than obtained by any of the other procedures described, and strengths obtained on specimens receiving no curing treatment were considerably lower.

Tests of the resistance to abrasion of the top surface (cured surface) of the specimens indicated that the use of 1½ per cent calcium chloride integrally and 1½ lb. per sq. yd. applied to the surface as soon as the bleeding water disappeared (no burlap) and all specimens cured by employing the surface application of calcium chloride whether in conjunction with burlap or not, resulted in a higher wear resistance than specimens cured with wet burlap applied for three days; however the wear resistance of specimens with liquid curing membranes or with no curing were somewhat less than the specimens cured with wet burlap applied for three days.

Maintenance Methods for Preventing and Correcting the Pumping Action of Concrete Slabs, Rex M. Whitton, Engineer of Maintenance, Missouri State Highway Department. This paper describes experience in Missouri with the correction of pumping at joints in concrete pavements by the use of a semi-fluid soil-cement mixture forced under the slab with a mudjack. The slurry used consisted of four sacks of cement per cubic yard of topsoil and 50 to 55 per cent of water. The spacing of the holes for elimination of pumping and correction of faulting is discussed. The equipment and personnel required is given in detail. On an average section of pavement, it was found that 354 holes per mile and 35.45 cu. yd. of slurry were required per mile of road. The average cost of the work was $24.78 for drilling 1½-inch holes and $256.66 per mile for the material and pumping operation.

In a study of the deflections of the pavement under moving loads, it was noted that deflections increased immediately after filling the voids under the pavement with the soil-cement slurry and then decreased after a period had elapsed. For a 12,000-lb. rear axle load, the deflections were reduced as much as 0.007 in. and for a 16,000-lb. wheel load as much as .011 in. between measurements made 9 days and 153 days after mudjacking.

It was found that a few of the mudjacked slabs resumed pumping and that the work had to be supplemented with joint and crack water-proofing to keep surface water from reaching the subgrade. On pavements that were cracked extensively, the best method of waterproofing joints was by the use of a substantial bituminous surface or upper deck not less than 1 inch in thickness.

After a study of design features from the viewpoint of a maintenance engineer, the author concludes that expansion or contraction joints should not be used in concrete pavements except at highway intersections, bridge ends or other locations where the pavement abutts a fixed object. The relation of the type of aggregate to the crack interval in concrete pavements in which no joints were placed is discussed.

Pumping of Concrete Pavements in New Jersey, Corrective Measures Employed, and Future Designs, William Van Breemen, Engineer of Special Assignments, New Jersey State Highway Department. Pumping at joints in concrete pavement slabs was first observed in New Jersey in 1930. It occurred on all pavement of standard design in which dowels ¾-in. in diameter were used for load transfer. A 100 per cent increase in the number of dowels did not eliminate the trouble. The use of crushed stone drains along the edge of the pavement was only partially effective in stopping pumping.

In 1932 a test road was built over a silty-clay soil. One joint with no load transfer device, two with six ¾-in. round dowels in the 10-foot width of pavement, two with twelve ¾-in. dowels, and several with various combinations of heavy rectangular and channel type dowels were placed in the slab. Continuous applications of heavy loads under adverse moisture conditions indicated that the use of a load transfer device composed of 2-in. channel-dowels was necessary to prevent faulting and subsequent pumping.

A recent survey of 60,000 channel-dowel joints on heavy duty highways disclosed only three failures that were caused by pumping. No faulting was found at these joints and the failures had occurred by sagging of the pavement. The stone drains along the edge of the pavement were partially clogged with subgrade soil.

A study of pavements laid on sub-bases composed of granular materials lead to the conclusion that their use minimized pumping, reduced damage due to frost action and increased load bearing capacity. All pavements built since 1939 are supported on a layer of bank-run sand, gravel or cinders 8 in. thick. To date, where granular materials have been used in conjunction with channel-dowel joints, pavements have remained true to grade, cracks are few and far between, and there have been no indications of pumping, even under heavy truck traffic.

A study of wood for use in expansion joints shows that for most varieties, loads in excess of 500 and less than 1000 psi will be required to cause compression of the fibers. If loading is continued, a point is reached where no further compression is obtained. Some varieties may be compressed to as much as 50 per cent of the original thickness. If dry wood is compressed to 50 per cent of its thickness, it will recover to about 65 per cent and remain at that thickness as long as it remains dry. Soaking in water will cause the wood to swell to 94 per cent, and for some varieties more than 100 per cent, of its original thickness. Repeated compression, drying and soaking will result in a permanent reduction in thickness. These tests indicate that wood as an expansion joint filler will have the following merits:

1. Unlike the conventional bituminous fillers, wood will not extrude, regardless of the extent of joint closure or infiltration. (This applies only to wood with the grain direction installed vertically).

2. Unlike other fillers, the wood is expected to retain sufficient swelling capacity and resiliency to prevent the detrimental accumulation and distribution of infiltrated material in the joint spaces which, in many locations, has caused rupturing of the concrete.

Investigation of Concrete Pavement Pumping, H. L. Krauser, Ohio Department of Highways. This paper describes the investigation of the pumping at transverse joints in concrete pavement slabs on a project 4.39 miles in length on U. S. 52 in Scioto County, O., near the village of Franklin Furnace.

The soils survey made prior to grading operations showed that on some sections the predominating soil was high in silt content and could be classified in the A-4 group. It was considered necessary to cover these areas with suitable granular material to a depth of 18 in. and provide tile drainage.

The pavement was built without reinforcing or load transfer devices. Records are also included on a short project built in 1941 in which complete reinforcing and load transfer devices were used. Observations were made of slab deflections under moving loads on both projects.

Pumping was found to be much more extensive and severe over the areas where no granular material was used. On the plain concrete section pumping was more severe at contraction joints of the premolded type. The severity on the reinforced section was about the same for expansion and contraction joints. On the plain concrete slabs without granular sub-base, 2 per cent of the expansion joints and 52 per cent of the contraction joints were pumping. On the plain concrete slabs placed on granular sub-base 1.5 per cent of the expansion and 16 per cent of the contraction joints were pumping. On the reinforced section, 71 per cent of the expansion joints and 91 per cent of the contraction joints were pumping.

On most of the work, the gradation of the backfill for the tile drains was from the ¾-in. to No. 4 sizes. It was observed that these drains silted up badly. On one section ⅜ in. to No. 8 material was used and only a small amount of silting was observed.

The conclusions of observations on these projects are as follows: 1. A sub-base composed of suitable granular material will appreciably reduce the pumping at joints in concrete pavements. 2. The use of small size backfill aggregate will extend the useful life of tile drains. 3. The use of load transfer devices prevents excessive permanent deformation at the joints between concrete slabs after pumping starts.

Use of Bituminous Materials as a Corrective Measure for Pumping Concrete Pavements, C. W. Allen and Harry E. Marshall, Ohio Department of Highways. The pumping of concrete pavement slabs on pavements in Ohio, which has developed since 1940, has followed the increase in volume of heavy truck traffic that has resulted from the concentration of war industries. A survey indicates that pumping has occurred mostly on soils of the A-4, A-6, or A-7 groups but is not confined exclusively to soils of these types. A study of the moisture contents of the subgrades at various depths indicates a maximum immediately beneath the pavements and a decrease with depth indicating that surface water is the chief source of the subgrade moisture that causes pumping. The use of transfer devices at joints and of granular subgrades have been found most effective in the prevention of pumping. The use of French drains in the shoulder was not effective in stopping pumping.

After experimenting with various soil-bituminous, portland cement mixtures and several grades of semisolid asphalts, it was found that an oil asphalt filler having a penetration of 30 to 45 at 77 F. was most satisfactory for filling the voids under pumping concrete slabs. This material is forced under the pavement by means of the hand spray equipment of a standard bituminous distributor through holes drilled in the pavement with a standard jackhammer and drill. The bituminous material forms a tight seal beneath the pavement and prevents the entrance of surface water and its stability is not affected by moisture from the subgrade. Although the costs of the asphalt is somewhat higher than any of the various soil-mixtures, a portion of this cost differential is equalized in the labor saved on the assembling and mixing of the various materials used in slurries.

Correcting Pavement Pumping by Mudjacking, R. E. Frost, Research Engineer, Joint Highway Research Project, Purdue University. This paper covers some field experi-

ments designed to correct the pumping action of rigid pavement slabs. In 1942 a performance survey on a portion of U. S. 30 between U. S. 41 and Valparaiso, Indiana was made by representatives of the Joint Highway Research Project covering detailed analysis of pumping conditions on twenty-four miles of this road. Among other things the results of the survey showed that all of the experimental subgrade treatments (with the exception of the water-saturated section) were successful in minimizing or eliminating pumping.

As a result of this survey, a series of joints in a two-mile section near the Lake-Porter County line were selected for treatment by mudjacking. Treatment of these joints was performed in October and November of 1942. Four mixes were used:

1. Mix A; 77 per cent soil, 7 per cent RC-3, 16 per cent cement
2. Mix B; 77 per cent soil, 7 per cent Road Oil, 16 per cent cement
3. Mix C; 77 per cent soil, 7 per cent Tar, 16 per cent cement
4. Mix D; 79 per cent soil, 3 per cent Tar, 17 per cent cement

Even though work was hampered by cold weather and numerous equipment breakdowns, a total of 434 cubic feet of mix were pumped under fifty-pumping joints in twelve-working days. This is an average of 8.7 cubic feet of mix per joint.

Several performance surveys of this two-mile section have been made since treatment to determine the permanence of the treatments. The most recent survey (Oct.-Nov. 1944) shows that pumping had been reduced considerably. However, the installation of subgrade drains on U. S. 30 between S. R. 49 and S. R. 53 together with a particularly dry year (1944) made it difficult to rate the success on the basis of pumping alone. The settlement at the joints of both treated and untreated slabs shows considerable success for mudjack treatment. It was found that the average settlement of the outer edges of treated slabs was 0.093 inches as compared to 0.194 inches for the untreated slabs. Further, it was found that 68 per cent of the treated slabs had settled one-eighth inch or less as compared to 53 per cent for untreated slabs.

A crack survey showed that mudjacking had been successful in reducing the expected number of cracks that would normally occur on this two-mile section. The data further show that cracks less than 13 feet either side of a joint are caused by slab movement and pumping and that cracks in the middle third of a slab are from causes other than pumping. Of the four mixes used, Mixes "A" and "D" contained less cracks and had less than 0.04 inch settlement.

The results of this two-year study show the desirability for additional research in mudjacking.

ACI NEWS LETTER

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The Institute's Forty-first Annual Convention

The forty-first Annual ACI Convention was the fourth of World War II. It was not the kind of annual convention to which the Institute looks forward in the peace years to come, nor the kind of convention which has made the annual meetings notable in former peace years.

The luncheon meeting at the Hotel New Yorker February 16, 1945 was much the same kind of meeting which characterized the 1943 convention in Chicago. This year seventy-two places were set (just one more than in Chicago in 1943); approximately two-thirds of those present were New York members. Those members in attendance who *travelled* to the meeting were almost exclusively confined to the more distant members of the Board of Direction and of the Advisory and Publications Committees.

There would undoubtedly have been a much larger attendance of local members but for unavoidable confusion about the restraints placed upon such gatherings under O. D. T. rulings. At first ACI had asked all New York members to attend if possible; then, facing a ruling that the *total* attendance at the New York meeting must be restricted to fifty (since the Institute had made no application for special dispensation) it was necessary to write to our New York members limiting the attendance of New Yorkers. Only in the last few days before the convention was it possible to advise local members in the region of the meeting that there was no restriction on their attendance, provided the attendance from a distance—the travelling membership—did not exceed fifty; this was the final O. D. T. ruling.

One pleasant ceremony was the award of Wason Medals by President Roy W. Crum, and unusual was the circumstance that in two instances the medals were received by ladies on behalf of their husbands.

Harrison F. Gonnerman, as previously announced in these pages, won the award for "Notable Research" reported in *Proceedings* V. 40 in his paper, "Tests of Concrete Containing Air Entraining Portland Cements—or Air Entraining Materials added to Batch at Mixer".

There being three authors of the paper chosen as the "Most Meritorious Paper", "Concrete Problems in the Construction of Graving Docks by the Tremie Method", there were, of course, awards in triplicate: To Commodore W. Mack Angas (CEC) U. S. N., Commander E. M. Shanley (CEC) U. S. N. R., and Lieutenant John A. Erickson (CEC) U. S. N. None of these medal winners could be present—all on distant Navy duty. The certificates of award were received by Mrs. Angas and by Mrs. Erickson for their husbands. Commander Shanley's award was received in his behalf by W. A. Durkin, Vice President, Walsh Construction Co., New York.

War conditions entered further into this ceremony in the fact that there were no medals for the winners of the awards for the "Most Meritorious Paper"—no bronze available from which to strike them off. So until bronze again becomes available for such purposes, winners of Wason medals will be receiving only the Certificate and their medals be deferred for postwar times.

Douglas E. Parsons became the new president of the Institute at the close of the New York meeting. He had served the Institute long and well, on both technical and administrative committees and as a member of the Board of Direction, and at many times arduously, and at some considerable personal sacrifice, there is reason to believe, on the Publications committee since 1940 and as chairman since 1941.

The report of the Tellers, R. R. Zipprodt and A. Burton Cohen, was presented by Mr. Zipprodt. All of the Nominating Committees' candidates for officers and directors were elected as follows:

President, Douglas E. Parsons, Chief of Masonry Construction Section, National Bureau of Standards, Washington, D. C.

Vice-President, Harrison F. Gonnerman, Manager, Research Laboratory, Portland Cement Association, Chicago, Ill.

Vice-President, Stanton Walker, Director of Engineering, National Sand and Gravel Association, Washington, D. C.

Director, First District (re-elected), Henry L. Kennedy, Manager, Cement Division, Dewey & Almy Chemical Co., Cambridge, Mass.

Director, Second District (re-elected), Myron A. Swayze, Director of Research, Lone Star Cement Corp., Hudson, New York.

Director, Third District, Alexander Foster, Jr., Vice-President, Warner Co., Philadelphia, Pa.

Director, Fourth District, Frank H. Jackson, Principal Engineer of Tests, Division of Physical Research, Public Roads Administration, Washington, D. C.

Director, Fifth District, Charles S. Whitney, Consulting Engineer, Milwaukee, Wis.

Director, Sixth District, Herbert J. Gilkey, Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa.

Director-at-Large, Robert F. Blanks, Chief, Engineering & Geological Control & Research, Bureau of Reclamation, Denver, Colo.

The five to serve as the elective members of the 1945 Nominating committee, with the three presidents last past, are as follows: R. B. Young, chairman; T. C. Powers, J. C. Pearson, T. E. Stanton, Frank H. Jackson. (Professor Gilkey, among the five receiving the most votes had immediately declined to serve).

In acknowledging his election to the Presidency of the Institute, Mr. Parsons had these things to say:

"I appreciate that this election implies some small amount of confidence in me. It is not important that I do not share that confidence because the Institute is not run by the President. As you know, our capable, energetic and overworked Secretary, together with the Board members and members of the committees actually do most of the running of the Institute.

"Perhaps it is less well known that individual members who receive no recognition for the part they play in the operation of the Institute do a great deal to guide the officers and to keep the policies and practices within proper bounds.

"One of the things that the officers will miss most this year will be the suggestions and criticisms from members. Sometimes those criticisms are expressed in most blunt and forceful language and often they are addressed to an officer at a time when the man making the suggestion is unaware that he is talking to the one who may be responsible for the the blunder that is being condemned. That has happened to me on a number of occasions during conventions.

"I wish that there was some way that we could continue to have that candid advice and criticism. The only way that I know that we shall have the benefit of it would be for each of you and the other members of the Institute to let us hear the worst with the best and say it in writing. thank you!"

President Crum acknowledged the presence at the meeting of Lt. Peter J. Doanides of the Royal Helenic Navy, Greece, on a special mission in this country, and Prof. Boris G. Skramtaiev, Doctor of Technical Sciences; head of a technical institute in Russia, and in this country attached to the Government Purchasing Commission of the U. S. S. R. Professor Skramtaiev has been a member of the Institute for nine years,

Douglas E. Parsons, newly elected President of the American Concrete Institute

a contributor to the ACI JOURNAL and has been in this country since May 1944 and expects shortly to be returning to Moscow. He had the following to say:

"It gives me great pleasure to be present here today, among leaders of the American Concrete Institute and its New York members. This is the first opportunity I have had to meet with American concrete men since 1936, the year when I joined the Institute.

"First, I would like to greet you cordially on behalf of all the Russian concrete specialists. We appreciate fully your achievements in the field of concrete, which you have developed so well. We very often use the results of your scientific work in our concrete practice in the Soviet Union, and the American names of Crum, Parsons, Abrams, McMillan, Gonnerman, Richart, and many others are famous among our Russian civil engineers.

"Meanwhile, we, in the Soviet Union, have also developed many different kinds of concrete, several new methods of winter work, and are making large quantities of precast reinforced concrete structures. We have also worked out a new theory for designing reinforced concrete, and now this theory, known as the Ultimate Theory, is the only official one in the Soviet Union.

"Next, I would like to take the liberty of suggesting that the new board of the Institute discuss the possibility of compiling a complete American bibliography pertaining to concrete. I am fully aware that this would be a difficult task, since you are already publishing very many books and articles, but I am sure you will agree that it is a very much needed piece of work.

"We have published a complete Russian bibliography on concrete, which I was glad to present to the American Concrete Institute. Upon my return to Moscow, I intend to continue this work in order to make this bibliography more complete by including all the Russian articles and books on concrete, cement and reinforced concrete constructions.

"It would seem to me highly desirable to establish a good-sized library at the American Concrete Institute for the use of all its members.

"May I close by wishing you the greatest success in all your work toward victory and also in the very extensive concrete jobs which you will be called upon to do after the war."

The address of President Crum in retiring from his office differed from previous presidential addresses. It had to do with the part of members of the Institute and others engaged in technical advancement with reference to our responsibilities in establishing a peaceful world in which such advancement may continue. The address appears on the first pages of this issue of the JOURNAL. Some who were present recall the address of Rear Admiral Ben Moreell (now Vice Admiral) on his retirement from the presidency of the Institute in 1942. The address was made from Washington by telephone and loud speaker hook-up at the convention, and sounded a note of responsibility of our membership in the war in which we became engaged such a short time before.

At a second meeting of the Board—the "new board"—following the luncheon meeting, several appointments were made. The election of Stanton Walker to a vice-presidency, one year before the completion of his three-year term as Director-at-Large left a vacancy to which Henry L. Kennedy, who had just been reelected Regional Director from the First District, was appointed. To fill the vacancy in the position of First District Director, the Board appointed Paul W. Norton, consulting engineer, Boston.

The resignation of Prof. Raymond E. Davis, after 6 years' service as chairman of the Advisory Committee, resulted in the election of Stanton Walker to fill that important place. Professor Davis remains on the committee in having been appointed to the chairmanship of Dept. 200.

On becoming President of the Institute, Mr. Parsons retired as chairman of the Publications Committee. Robert F. Blanks, Bureau of Re-

clamation, a member of the Board as Director-at-Large, was appointed to that very active committee.

The Board also appointed Harvey Whipple to succeed himself as Secretary-Treasurer.

WHO'S WHO in this ACI JOURNAL

Jacob Feld

ACI Member since 1944, presents his first paper (p. 441). Dr. Feld is a consulting civil engineer, New York City. He was born in Austria, attended the College of City of New York (B.S. 1918); attended Univ. Cincinnati (C.E. 1920, M.A. 1921, Ph. D. 1922, Mem. Phi Beta Kappa, Sigma Xi, Sigma Tau Phi).

In New York he was employed by the Public Service Comm., Queens Highway Dept. and Erie R.R. From 1919-1922 he was a Research Fellow, Institute C. E., Univ. of Cincinnati; in 1922 with Turner Construction Co., N.Y.; 1923, Long Island, R. R. as assistant bridge engineer; 1924, with Robinson & Steinman, bridge engineers; 1925, Barney-Ahlers Co., as concrete Engineer; 1925 to date, Consulting Engineer, bridges, buildings. His work has ranged through foundations, subways, radio towers, stadiums, dairy plants, an average of $5,000,000 work per year. He was consulting engineer for the contractors on the Tri-borough Bridge contracts, 205 St. Viaduct, grade crossing eliminations, bridge foundations, 1930-1935. He was in charge of construction of section 10 of the Sixth Ave. Subway for Brader Construction Corp. 1936-1939. He has done research in lateral earth pressure, foundations, tower analysis, long span cables, stadium designs, 1000 ft. guyed radio towers at Shanghai; is author of "Lateral Earth Pressure," A.S.C.E. Trans. 1923, "History of Earth Pressure Theories," Brooklyn Engineers Club *Proc.* 1929, "Unbraced Cables" Franklin Institute 1930 and numberous other papers. He is a member of A.S.C.E. (Collingwood Prize 1923), (F.), A.A.A.S., Member Brooklyn Engineers Club (A. T. White Prize 1928) Dir. 1927-1933, and of New York Academy Sciences.

D. R. Cervin

member of the Institute since 1943 presents his first Institute paper (p. 453). Mr. Cervin received his B.S. degree in architectural engineering from the University of Illinois in 1934; did graduate work in structural engineering at Illinois in 1936; and at the University of Iowa in 1937. He was with the United States District Engineer Office, Rock Island, Ill. 1934 to 1938 in connection with the design and construction of the locks and dams for the 9 ft. channel in the Mississippi River. In 1939 he was with A. H. Ebeling, Davenport, Iowa, architect, doing architectural engineering work on school buildings; in 1940 he was engaged in construction of the spill-way for the Santee Cooper project in South Carolina as office engineer for the McCarthy Improvement Co., and for the next year and a half he was with the special engineering division in the Canal Zone on the design of the third locks, a project which was terminated by the war. Another year and a half was spent with the United States Army District Engineer office in Jacksonville, Fla. as assistant head of the structural design section; his work included army camps, airports and appertenant works in the Florida area. Now, and for the past year he has been employed as a structural engineer for the National Advisory Committee for Aeronautics and his work includes the design and construction of the buildings required by the expanded research necessary for improving the field of aeronautics.

Gerald Pickett

who unfortunately we cannot claim as an ACI Member, still is not new to these pages. To the Feb. 1942 JOURNAL he contributed "The Effect of Change in Moisture Content on the Creep of Concrete under a Sustained Load". At that time it was noted in these columns that he was a graduate electrical engineer from Oklahoma Agricultural and Mechanical College in 1927; awarded a Ph.D. in mechanics by the University of Michigan in 1938; and aside from two years in the testing laboratory of the Brooklyn Navy Yard, he taught in the Department of Mechanics, Kansas State College until 1940—advanced courses in mechanics for graduate students in addition to regular courses to undergraduates. He joined the basic research division of the Portland Cement Association Research Laboratory, Chicago in 1940 as a research physicist, studying the mechanics of portland cement concrete.

Dr. Pickett has been interested in problems involving dynamics for several years and his paper on the "Dynamic Testing of Concrete" (p. 473) is a natural outgrowth of his earlier work in other fields of mechanics.

Jacob J. Creskoff

member of the Institute since 1937, is not new to these pages. To volume 33 he contributed "Earthquakes and Reinforced Concrete". At that time it was noted in this column that he was the author of "Dynamics of Earthquake Resistant Structures" (McGraw-Hill Book Co.), that he had worked for various Government agencies on aseismic design and that he had been consultant to the Treasury Department on the design of a new earthquake resistant San Francisco mint. He is a graduate from the civil engineering department Towne Scientific School, University of Pennsylvania. From 1937 to 1942 he was vice president of Vacuum Concrete Corp., Philadelphia; and from 1942 to date, resident vice president, McCloskey & Co., Washington D. C. and Tampa, Fla; actively identified with the McCloskey work on concrete ships. His present paper, as a contribution to the prediction of the strength of concrete appears p. 493.

Inge Lyse and Family escape from Norway

The many Institute friends of Prof. Inge Lyse will be pleased to know that he and his family who had been at Trondheim, Norway for several years, late in 1944 escaped with only what they could carry on their backs over the mountains into Sweden where they are safe with friends.

This news reached a number of Institute members at the time of the Institute's meetings in New York in February by way of a copy of a letter which Mrs. Lyse had written to friends in Bethlehem, Pa.

Returning from the New York meetings the Secretary of the Institute found a letter addressed to him January 9 from Stockholm in which Professor Lyse wrote "just a few words to let you know that I am here in Stockholm with my family and that we are all well. I had a narrow escape from the Gestapo, but the trip across the mountains solved the problem. I am at present working at the Swedish Cement and Concrete Research Institute, which has just moved into its new building at the Royal Technical Institute". He reports that his time with the Swedish Institute will be limited to that which he can spare from his engagement at the Norwegian Military Offices in Stockholm; "with best greeting to all my friends in the ACI".

More detailed and revealing of the times through which the Lyses had passed, since the German invasion of Norway, where Lyse was identified with Norway's Institute of Technology at Trondheim, are a few lines from Mrs. Lyse's letter which was written Dec. 26:

"The boys have been ill in bed since we came here, but are now perfectly all right again—their stomachs were unused to the rich food we get here, and besides meat, white bread and cheese and jam, they have been getting candy and chocolate, which was too much for their stomachs; but now we can eat everything without getting sick—and do we eat . . . Norway is absolutely emptied of all goods; and the joy to see lights again; you can't imagine what that means after having lived in a country constantly blacked out."

Professor Lyse, active member of ACI since 1926, three years after coming to the United States (see ACI Directory 1944, p. 70), in 1938 left Lehigh University, where he was in charge of Fritz Engineering Laboratory, to return to Norway's Institute of Technology, Trondheim, his alma mater, to be professor of reinforced concrete and solid bridges (see ACI Jl. N. L. p. 14, Sept. 1938).

Leo Nagel

Only recently did the Institute learn of the death of Leo Nagel, member of the Institute since 1939; resident of Detroit. He was a graduate of the Technical University of Vienna, Austria, 1915, and for 15 years before coming to America, he was managing director of one of the large engineering construction companies of Vienna, specializing in reinforced concrete in the Industrial field. In 1940, coming to the United States, he entered the organization of Albert Kahn, Inc., Detroit, where he was employed until the time of his death in August 1944.

The June 1945 JOURNAL will be notable in containing by direction of the Board all the ACI Standards promulgated since the inauguration of the present Standards Committee in 1937.

Alfred E. Lindau

The following is from the minutes of a meeting of the Institute's Board of Direction, New York, February 14 (See N.L. January, 1945):

"In the death of Alfred E. Lindau at Pearl Harbor, Hawaii, December 14, 1944, the American Concrete Institute lost one of its best known and consistently active workers. For three and a half decades he had been a leader in the thought and activities of the Institute, few of whose members had contributed so much over so many years from such a wide range of activity; few indeed, had been held in greater esteem by their fellow members for a high degree of intellectual integrity and a well stocked and finely discriminating mind.

"For five years (1939-1944) he had given strenuously and generously of his engineering knowledge to the war effort as Chief Civilian Engineering Assistant to Officer-in-Charge, Pacific Naval Air Bases.

"Alfred Lindau joined the Institute in 1909, was elected an Honorary Member in 1935. He had been a member of the Board of Direction, as a director in 1915 and 1916 and continuously a Board member from 1922 to 1935—as vice president, 1922 to 1923, president, 1924 and 1925, past president member 1926 to 1935. In returning to the ranks he continued a willing worker on many important tasks. He was unexcelled as an Institute convention presiding officer, bringing to the leadership of discussion not only his broad knowledge, but his dynamic spirit and a contageous interest.

"His lively interest in a wide range of human experience, his quick and deep understanding, his rare combination of objective judicial-mindedness with great warmth of personality and unfailing kindness won him the high respect and affection of a host of friends."

Honor Roll

February 1, 1945 to March 29, 1945

In the February issue we reported the 1945 Honor Roll only to January 29 and there were changes in the two days to January 31 which ended the scoring year.

Lewis H. Tuthill, top man, finished with a final score of 18 Members sponsored and F. E. Richart 13. H. F. Gonnerman increased his score from 2½ to 3½ and Jacob Fruchtbaum's name was added to the roll in his sponsoring one new member.

For several years the two top men each year have earned a worthwhile credit, as a Board of Direction award, for engineering literature purchased through the Institute.

By recent Board action the rules are changed and the two top men are to receive Certificates of Merit for important service to the Institute. The first entries, for the Certificate to be presented at the 1946 Convention (for the year ending January 31, 1945) are as follows:

A. Amirikian	3
J. H. Spilkin	3
C. Blaschitz	2
Charles S. Whitney	2
R. F. Dierking	1
Charles E. Wuerpel	1
Wm. G. McFarland	1

50-50 With Other Member:

Kanwar Sain
J. L. Savage

Subject to the possible exigencies of war, the Institute's 42nd annual convention is scheduled to be held in New York City, the week of February 18—a "full-blown" convention.

New Members

The Board of Direction approved 49 applications for Membership (43 Individual, 3 Junior, 3 Student) in January and February as follows:

Anderson, Boyd G., 4336 Livingston Rd., S.E., Washington D. C.

Arena, de la, Pedro Guash, Calle O esq. 25, Edit. Castro, Vedado, Havana, Cuba

Arnal M., Dr. Eduardo A., P. O. Box 263, Caracas, Venezuela, S. A.

Atkins, Clinton P., 115 Victory Lane, Leetsdale, Pa. (Student)

Atkinson, James L., P. O. Box 482, National City, Calif.

Aydin, Abdurrahman, U. P. O. Box 572, Princeton, N. J.

Barker, Austin, c/o U. S. Bureau of Reclamation, Mancos, Colo.

Bauman, E. W., National Crushed Stone Assoc., 1735 14th St. N. W., Washington, D. C.

Chatterley, Jay, 795 N. 8 E., Logan, Utah (Student)

Croll, Donald, 74 Hedley Place, Buffalo 8, N. Y.

Dominguez C., Dr. Atahualpa, Garita a Pepe Aleman No. 12-4, Caracas, Venezuela, S. A.

Evans, I. G., Building Research Station, Garston, Watford, Herts, England

Ewart, Raymond R., 2660 Quaker Bridge Rd., Box 45, Mercerville, N. J.

Finney, E. A., Michigan State College, Rm. 3, Olds Hall, East Lansing, Mich.

Furnival, George E., c/o Furnival-Rimmer Co., 54th and Lancaster Ave., Philadelphia 31, Pa.

Hom, Harry Goff, 4051 Front St., San Diego 3, Calif. (Junior)

de Lys-Gregson, H. St. J. R., c/o Messrs. Grindlay & Co. Ltd., 6 Church Lane, Calcutta, India

Handa, C. L., c/o The India Supply Mission, 635 F. Street, N. W., Washington, D. C.

Hansen, Waldemar C., c/o Universal Atlas Cement Co., Gary, Buffington Station, Ind.

Haws, Frank W., 37 E. 3rd So., Logan, Utah (Student)

Hepplewhite, Lt. W. T., c/o Director of Fortifications & Coastal Works, P. O. Box 2333, Cape Town, So. Africa

Holdampf, Carl R., 5844 N. Shoreland Ave., Milwaukee 11, Wisc.

Holm, J. C., c/o National Portland Cement Co., Brodhead Plant, R.F.D. No. 1, Bethlehem, Pa.

Howland, L. D., 1863 Meadowbrook Rd., Altadena, Calif.

Kaplan, Sgt. M. F., c/o Director of Fortifications & Coastal Works, P. O. Box 2333, Cape Town, So. Africa

Kleinman, Sgt. S., c/o Director of Fortifications & Coastal Works, P. O. Box 2333, Cape Town, So. Africa

Luke, Irvin H., Apartado 4068, San Juan, P. R.

Martin, Ramon Barcelo, Espada No. 55, Apt. 22, Havana, Cuba

Mielenz, Richard C., Petrographic Laboratory, Bureau of Reclamation, New Customhouse, Denver 2, Colo.

Moore, Thomas, c/o Harrop Chemical Co., 135 Hoboken Ave., Jersey City, N. J.

Morris, Max H., 8714 Cameron St., Silver Spring, Md. (Junior)

Nelson, H. T., 7½ N. 2nd St., c/o U. S. Bureau of Reclamation, Yakima, Wash.

Newlands, James A., 11 Saurel St., Hartford, Conn.

Odley, Ezra G., 4801 Connecticut Ave., Washington 8, D. C.

Olsen, William G., 406 Building & Loan Bldg., Raleigh, N. Car.

Plummer, Harry C., c/o Structural Clay Products Institute, 1756 K Street, N. W. Washington 6, D. C.

Ravitch, Rosalyn, 15 West 81 St., New York 24, N. Y. (Junior)

Reville, James F., 1448 18th St., S. E., Washington 20, D. C.

Robinson, Edward A., 194 Read Ave., Tuckahoe, N. Y.

Rubinsky, Moe A., 2348 E. Ave., National City, Calif.

Salabarria, Orestes Vergara, Calle 13, entre 9a and 10a, Ampl. Almendares, Marianao, Havana, Cuba

Salgo, Michael N., 228 N. Thomas St., Arlington, Va.

Sletholt, C. Henry, 8838 53 Street, Brooklyn, N. Y.

Thorson, Comdr. E. W., c/o Bureau of Yards & Docks, Navy Department, Rm. 4426, Washington, D. C.

Turley, Sylvester J., 7 22nd Lane, National City, Calif.

VanPetten, R. M., 5509 Lakeshore Drive, Knoxville 15, Tenn.

Ward, C. N., 550 State St., Madison 3, Wisc.

Wickwire, J. L., Asst. Chief Engr., Dept. of Highways & Public Works, Halifax, Nova Scotia

Witter, Harry C., 3059 Seaview Rise, Honolulu, T. H.

(*Adv.*)

WANTED—Superintendent for concrete products plant specializing in Architectural Concrete Slabs, Cast Stone, Pre-Cast joists and Specialty Products, must be versed in blue print reading, concrete mix designs, production schedules and capable of handling men. Will offer interest in business to right man. Annual volume well over quarter million dollars. Please state age, previous experience and salary desired. Address Box AA, c/o American Concrete Institute, Detroit 2, Mich.

See June 1945 for all ACI Standards adopted since 1937.

Sources of Equipment, Materials and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 45th St., Alpena, Mich. 428
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 387
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment

Baily Vibrator Co., 1526 Wood St., Philadelphia 2, Pa. 407
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 394-5
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 421
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 418-9
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 410-11
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 389
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 377
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 378-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 392-3
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 429
—Mixing and batching plants, buckets, elevators

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 422-3
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 380
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 427
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 430-1
—Concrete vibrators

—Continued next page

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Ransome Machinery Co., Dunellen, N. J. 433
—Mixers—paving, truck

Viber Co., 726 So. Flower St., Burbank, Calif. 381
—Concrete vibrators

Contractors, Engineers and Special Services

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 390-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 397-400
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 396
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 426
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 416
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 409
—Suction control of water in the concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 385
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge, Mass. 412-5
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 417
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 425
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 384
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 382-3
—Portland cements

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 402-5
—Cement dispersing and air-entraining agents

Rail Steel Bar Association 386
—Reinforcement bars

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 408
—Form tying devices

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 434-5
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 401
—Form lining

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American Machine & Metals, Inc., East Moline, Ill. 388
—Riehle hydraulic testing machines

[To facilitate selective distribution, **separate prints** of this title (41-22) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1945.]

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Slabs Supported on Four Sides*

Suggested Changes in ACI Building Regulations

By R. L. BERTIN, JOSEPH DI STASIO, and M. P. VAN BUREN†
Members American Concrete Institute

SYNOPSIS

The ACI Building Regulations for Reinforced Concrete provide, with respect to slabs supported on four sides, a method of analysis which reflects a clear picture of the elastic action of the structure, and, through the use of equivalent uniform load factors, permits the direct solution for bending moments and shears in the slabs and beams with the same coefficients as prescribed for one-way construction. To clarify the manner of presentation, the authors have prepared a suggested change of the entire Chapter 7 of the Code. While retaining all the original basic features, notation has been simplified, non-essential formulas and extraneous theory eliminated, and the regulations condensed to the fundamentals requisite for direct design. Final results are unchanged from those obtained through the use of the present 1941 regulations.

In this paper, the proposed changes are stated and reasons for them given. Suggested regulations are presented in new form. Comparisons are shown to indicate conformity with theory. Finally, an analysis of a typical series of floor panels is given to illustrate the facility with which computations could be made under suggested changes. It is believed that engineers would find the suggested modification of this section of the Code simple and easy to apply.

INTRODUCTION

The development of empirical methods of design for inclusion in building codes is generally motivated by two basic concepts stated in the order of their importance to the designers:

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†R. L. Bertin, Chief Engr. White Construction Co., Joseph Di Stasio, and M. P. Van Buren, J. Di Stasio & Co., Consulting Engrs., N. Y. City.

(a) Simplicity of application,

(b) Adherence to theoretical correctness.

This is particularly true of two-way slab design because of the complexity of the theoretical analysis.

It is believed that the ACI regulations yield results throughout the range of two-way slabs conforming more nearly than any other to the theoretical analysis, and in the new suggested form it is not only simple of application, but compatible with the frame analysis of continuous structures in which two-way slabs occur.

It is the purpose of this paper to present comparative data from which these beliefs are deduced. Sub-divisions of the present paper are: Part 1—Suggested modified regulations; Part 2—Comparative analysis: Part 3—General application.

PART 1

Suggested changes

The regulations first developed for the New York City Building Code were recommended to the ACI by Committee 501 in its Proposed Building Regulations in 1935, and the basic theory explained by a paper in 1936*.

The original 1935 transcript was re-arranged and amplified by tables in the ACI Building Regulations for Reinforced Concrete, adopted in 1941. The re-editing of this section as suggested by the authors, includes the following proposed modifications:

1—Elimination of tables and formulas involving determination of lines of inflection. Elastic analysis is a definitely established theory and requires no special exposition under this heading. Limitations are retained within which prescribed values of the distance between inflection lines may be used.

2—Elimination of special treatment of unusual cases by definitely specifying that all slabs be securely attached to supports. Structures in which panel edges are free to uplift are of limited practical value and should be considered as individual problems.

3—Simplification of formulas for moment and shear. Terms involving width of slab strip and tributary width carried by beam, are eliminated by expressing the load in terms of W, the total slab load on either slab strip or beam. All factors, by which moment and shear at any section in one-way construction in either direction are obtained, are presented in two short tables. Corresponding charts are also shown as graphic illustrations of the tables.

*"Slabs Supported on Four Sides," by Di Stasio and Van Buren, ACI JOURNAL Jan.-Feb. 1936; *Proceedings* V. 32, p. 350.

4—Simplification of notation. Special symbols are eliminated and replaced with a condensed and more familiar nomenclature. Of these, some, such as L and W may be used without change in definition for other sections of the Code.

5—Elimination of all footnote formulas. These are no longer required as all information for the design of slabs may be obtained from the tables in their suggested form. For general information, the formulas on which the tables are based are given here as derived from the 1941 regulations by direct substitution in the new notation.

$$r > .5.C = \left(\frac{1}{1+r^3}\right)\left(\frac{2r.}{4r-1}\right) \quad \text{.....(1)}$$

$$r < .5.C = \frac{1}{1+r^3} \quad \text{.....(2)}$$

$$C_a = (.5 - X)\left(\frac{1 - 6X}{1 + r^3} + 6XC\right) \quad \text{.....(3)}$$

$$C_b = .5 - X - C_a \quad \text{.....(4)}$$

Suggested regulations

700—Notation (For Slabs Supported On Four Sides)

L = Span length

L_1 = Span length in the direction normal to L in floors supported on four sides.

m = Ratio of span between lines of inflection to L in the direction of span L, when span L only is loaded.

m_1 = Ratio of span between lines of inflection to L_1 in the direction of span L_1, when span L_1 only is loaded.

r = Degree of rectangularity between lines of inflection of a panel supported on four sides = $\dfrac{mL}{m_1L_1}$

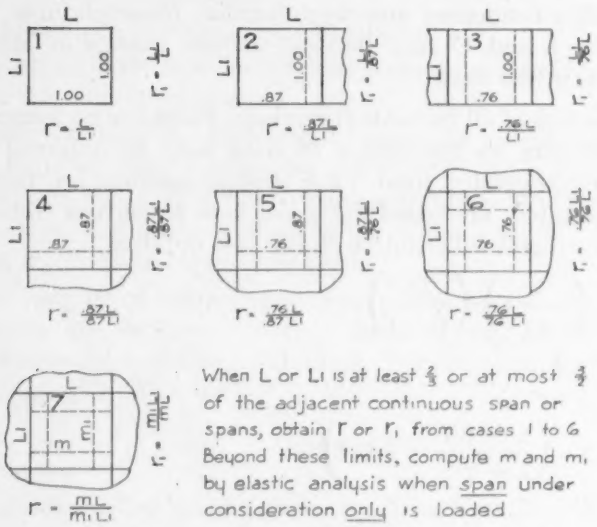
w = Uniformly distributed total load per unit area of slab.

W = Total uniform load for one way construction between opposite supports on slab strip of any width or on beam in the direction of L.

X = Ratio of distance from support to any section of slab or beam, to span L or L_1.

C = Factor modifying bending moments prescribed for one-way construction for use in proportioning the slabs and beams in the direction of L of slabs supported on four sides.

C_a = Ratio of the shear at any section of a slab strip distant xL from the support to the total load W on the strip in direction of L.



When L or L1 is at least $\frac{2}{3}$ or at most $\frac{3}{2}$ of the adjacent continuous span or spans, obtain r or r_1 from cases 1 to 6. Beyond these limits, compute m and m_1 by elastic analysis when span under consideration only is loaded

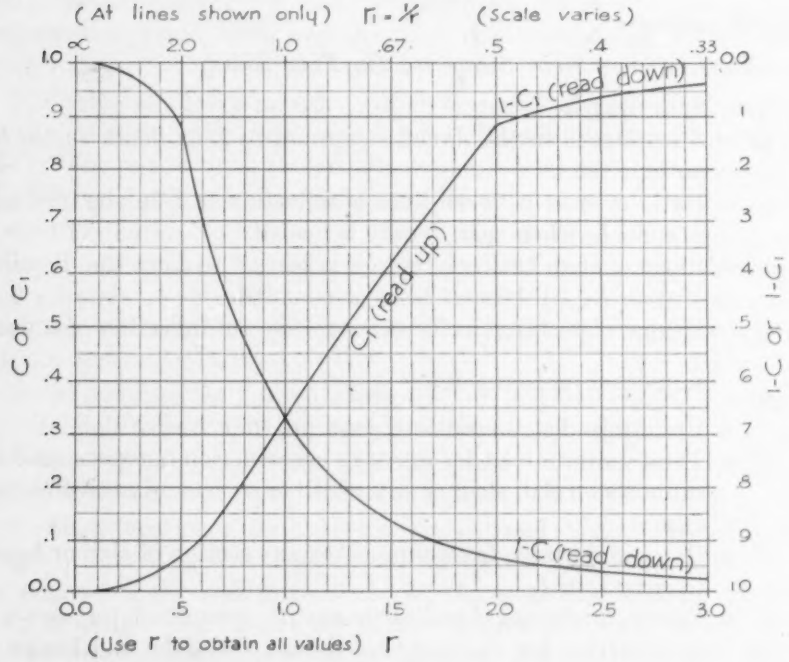

(At lines shown only) r1 = 1/r (Scale varies)
∞ 2.0 1.0 .67 .5 .4 .33 0.0
1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1 0.0
1-C1 (read down)
C1 (read up)
1-C (read down)
C or C1
1-C or 1-C1
0.0 .5 1.0 1.5 2.0 2.5 3.0
(Use r to obtain all values) r

Fig. 1 (at top) and 2

C_b = Ratio of the Shear at any section of a Beam distant xL from the support to the total load W on the Beam in direction of L.

$B.M.C.$ = Bending Moment Coefficient.

W_1, C_1, C_{s1}, C_{b1}, are corresponding values of W, C, C_s, C_b, for slab strip or beam in direction of L_1.

709—Floors supported on four sides

(a) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with burnt clay or concrete tile fillers, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The slab shall be supported around its periphery along the spans L and L_1, and shall be securely attached to said supports by monolithic construction or other adequate means.

(b) *Minimum slab thickness*

The slab thickness shall satisfy prescribed working stresses and shall not be less than 4″ nor less than the sum of the clear length of all supports at which the slab is continuous with the adjacent panels divided by 180 plus the clear length of all other supports divided by 144.

(c) *Lines of inflection for determination of r*

The lines of inflection shall be determined by elastic analysis of the continuous structure in each direction, when the span under consideration *only* is loaded.

When the span L or L_1 is at least 2/3 and at most 3/2 of the adjacent continuous span or spans, the values of m or m_1 may be taken as 0.87 for exterior spans and 0.76 for interior spans. (See Fig. 1).

For freely supported spans m or m_1 shall be taken as unity.

(d) *Bending moments and shear*

Bending moments shall be determined in each direction with the coefficients prescribed for one-way construction in section 701 and 702 and modified by factor C or C_1 from Tables 1 or 2 or from chart. (See Fig. 2).

	In L Direction	*In L_1 Direction*
B. M. for slab strip	$= M = CWL\ (B.M.C.)$	$M_1 = C_1W_1L_1\ (B.M.C.)$
B. M. for beam	$= M = (1—C)\ WL\ (B.M.C.)$	$M_1 = (1—C_1)\ W_1L_1\ (B.M.C.)$

When the coefficients prescribed in 701(c) are used, the average value of Cw or C_1w for the two spans adjacent to a support shall be used in determining the negative bending moment at the face of the support.

The shear at any section distance xL or xL_1 from supports shall be determined by modifying the total load on the slab strip or beam by the factors C_s, C_{s1}, C_b *or* C_{b1} taken from Table 1 or 2 or from charts. (See Fig. 3 or 4).

TABLE 1—SLABS

Upper Figure / Lower Figure			C_s / C_{s1}					C / C_1
r.	$\frac{1}{r}$	X	0.0	.1	.2	.3	.4	
0.00	∞		.50 .00	.40 .00	.30 .00	.20 .00	.10 .00	1.00 .00
.50	2.00		.44 .06	.36 .03	.27 .02	.18 .00	.09 .00	.89 .06
.55	1.82		.43 .07	.33 .04	.23 .02	.15 .01	.07 .00	.79 .08
.60	1.67		.41 .09	.30 .05	.20 .03	.12 .01	.05 .00	.70 .10
.65	1.54		.39 .11	.28 .06	.18 .03	.10 .01	.04 .00	.64 .13
.70	1.43		.37 .13	.26 .08	.16 .04	.09 .01	.03 .00	.58 .15
.80	1.25		.33 .17	.22 .10	.13 .06	.07 .02	.02 .00	.48 .21
.90	1.11		.29 .21	.19 .13	.11 .07	.05 .03	.01 .01	.40 .27
1.00	1.00		.25 .25	.16 .16	.09 .09	.04 .04	.01 .01	.33 .33
1.10	.91		.21 .29	.13 .19	.07 .11	.03 .05	.01 .01	.28 .39
1.20	.83		.18 .32	.11 .21	.06 .13	.02 .06	.00 .02	.23 .45
1.30	.77		.16 .34	.10 .23	.05 .14	.02 .07	.00 .03	.19 .51
1.40	.71		.13 .37	.08 .25	.04 .16	.02 .09	.00 .03	.16 .57
1.50	.67		.11 .39	.07 .27	.04 .17	.01 .10	.00 .04	.14 .61
1.60	.63		.10 .40	.06 .29	.03 .19	.01 .11	.00 .05	.12 .66
1.80	.55		.07 .43	.04 .33	.02 .23	.01 .15	.00 .07	.08 .79
2.00	.50		.06 .44	.03 .36	.02 .27	.00 .18	.00 .09	.06 .89
∞	0.00		.00 .50	.00 .40	.00 .30	.00 .20	.00 .10	.00 1.00

TABLE 2—BEAMS

Upper Figure / Lower Figure			C_b / $C_{b'}$					$1-C$ / $1-C_1$
r	$\frac{1}{r}$	X	0.0	.1	.2	.3	.4	
0.00	∞		.00 .50	.00 .40	.00 .30	.00 .20	.00 .10	.00 1.00
.50	2.00		.06 .44	.04 .37	.03 .28	.02 .20	.01 .10	.11 .94
.55	1.82		.07 .43	.07 .36	.07 .28	.05 .19	.03 .10	.21 .92
.60	1.67		.09 .41	.10 .35	.10 .27	.08 .19	.05 .10	.30 .90
.65	1.54		.11 .39	.12 .34	.12 .27	.10 .19	.06 .10	.36 .87
.70	1.43		.13 .37	.14 .32	.14 .26	.11 .19	.07 .10	.42 .85
.80	1.25		.17 .33	.18 .30	.17 .24	.13 .18	.08 .10	.52 .79
.90	1.11		.21 .29	.21 .27	.19 .23	.15 .17	.09 .09	.60 .73
1.00	1.00		.25 .25	.24 .24	.21 .21	.16 .16	.09 .09	.67 .67
1.10	.91		.29 .21	.27 .21	.23 .19	.17 .15	.09 .09	.72 .61
1.20	.83		.32 .18	.29 .19	.24 .17	.18 .14	.10 .08	.77 .55
1.30	.77		.34 .16	.30 .17	.25 .16	.18 .13	.10 .07	.81 .49
1.40	.71		.37 .13	.32 .15	.26 .14	.18 .11	.10 .07	.84 .43
1.50	.67		.39 .11	.33 .13	.26 .13	.19 .10	.10 .06	.86 .39
1.60	.63		.40 .10	.34 .11	.27 .11	.19 .09	.10 .05	.88 .34
1.80	.55		.43 .07	.36 .07	.28 .07	.19 .05	.10 .03	.92 .21
2.00	.50		.44 .06	.37 .04	.28 .03	.20 .02	.10 .01	.94 .11
∞	0.00		.50 .00	.40 .00	.30 .00	.20 .00	.10 .00	1.00 .00

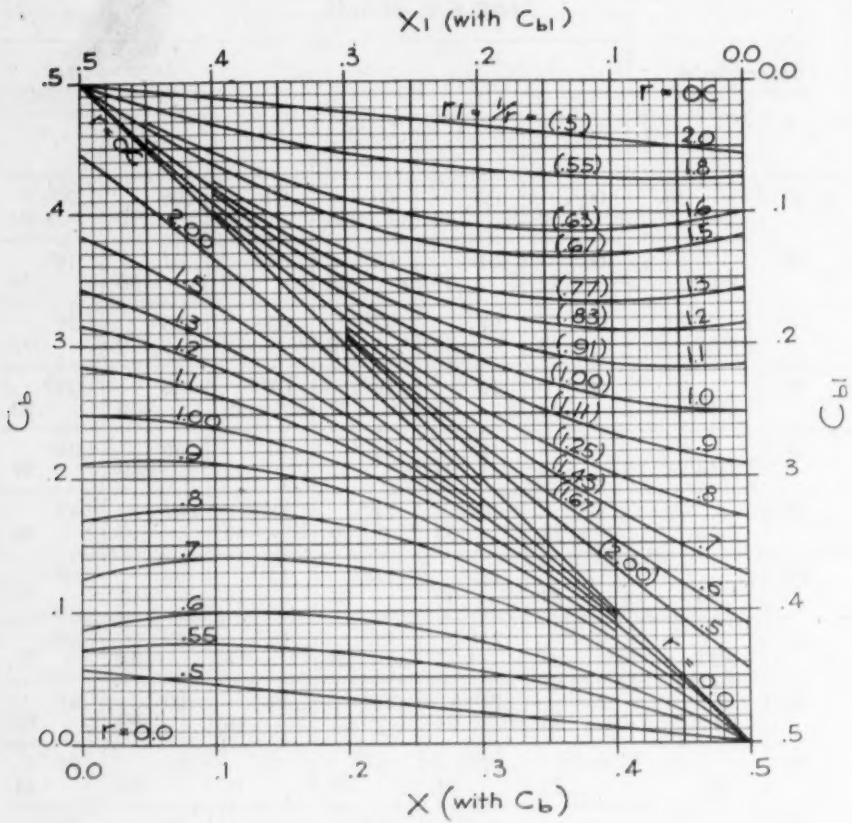


Fig. 3

	In L Direction	*In L_1 Direction*
Shear for Slab Strip	$V = C_s W$	$V_1 = C_{s1} W_1$
Shear for Beam	$V = C_b W$	$V_1 = C_{b1} W_1$

For spans where the end moments are unbalanced, shear values at any section shall be adjusted in accordance with Sections 701 and 702.

(e) *Arrangement of reinforcement*

1. In any panel, the area of reinforcement per unit width in the long direction shall be at least one-third that provided in the short direction.

2. The area of positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.

3. At a non-continuous edge the area of negative moment reinforcement per unit width shall be at least-one-half of that required for maxi-

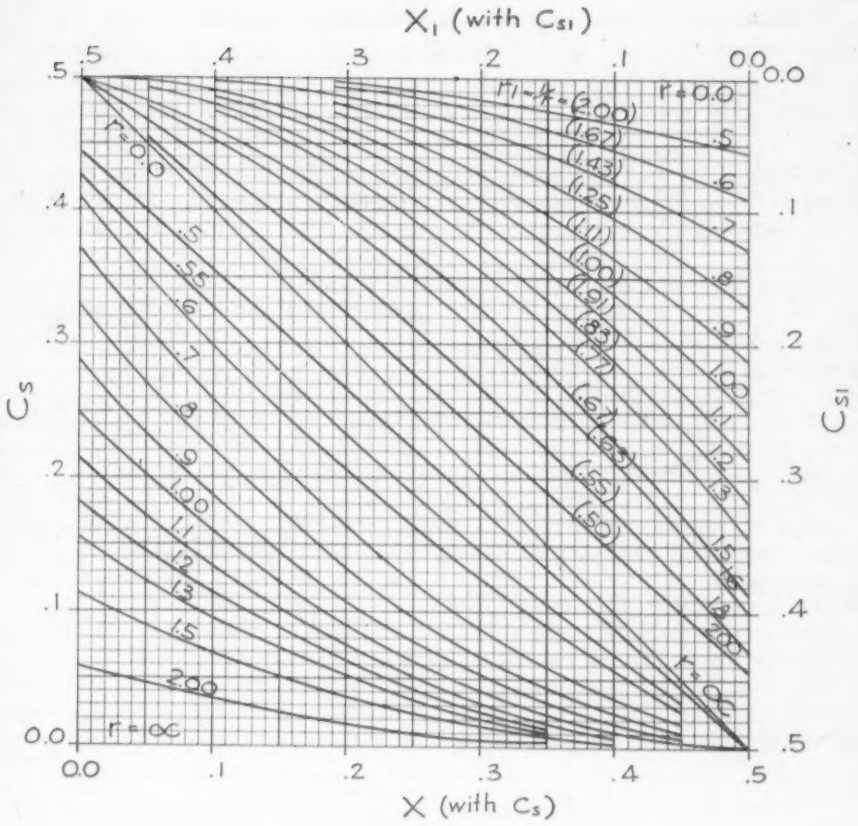


Fig. 4

mum positive moment for the center one-half of the panel and shall be provided across the entire width of the exterior support.

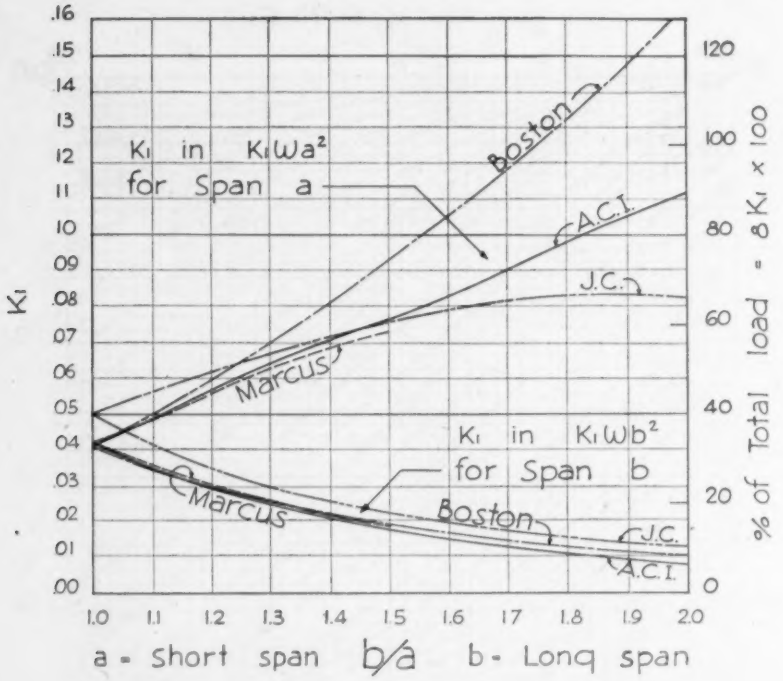
4. The spacing of the bars shall be at most three times the slab thickness and the ratio of reinforcement at least 0.0025.

PART 2

Conformity with theory

This section is devoted to a comparison of these regulations with the results of other authoritative investigations and codes, namely, the formulas of Dr. Marcus*, Dr. Westergaard†, the 1940 Joint Committee, and the Boston Code. In order to eliminate any difference in primary assumptions as to proportion or arrangement of the live load, panels

*"Design of Reinforced Concrete Slabs," Joseph A. Wise, ACI *Proceedings*, V. 25, 1929.
†"Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders," H. M. Westergaard, ACI *Proceedings* V. 22, 1926.



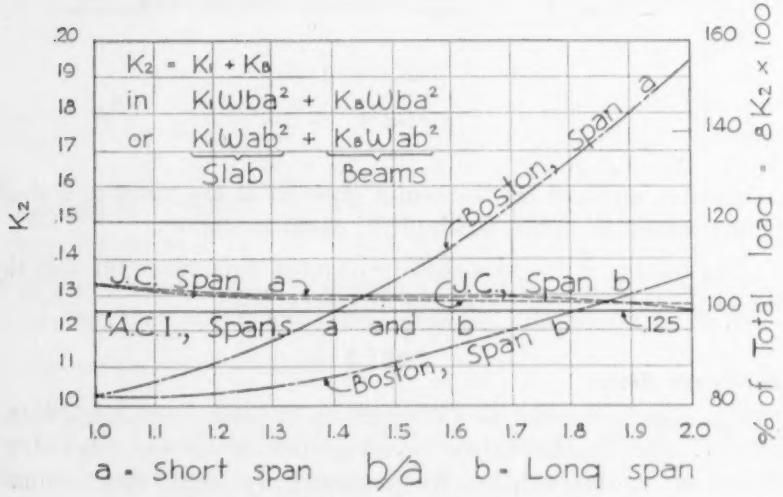


Fig. 5 (top)—Slab moment coefficients in simple panel

Fig. 6—Sum of moment coefficients for parallel beam and slab in simple panel

with various combinations of free and fully fixed edges have been selected as representing the extreme range of continuity for one definite loading condition. As indicated on the graphs of Fig. 5 to 8, the ACI Code more nearly conforms to the theoretical analyses of two-way slabs than any other code, as follows:

(a) Load Distribution. The equivalent uniform load causing slab bending ($8K_1w$) in either direction of a simply supported panel agrees closely with the results of Dr. Marcus, which, in the opinion of the authors, present the most direct theoretical approach to this problem. (See Fig. 5).

(b) Total Panel Moment. The sum of the beam and slab moments in either direction (K_2WL) equal the statical moments (1/8WL). (See Fig. 6).

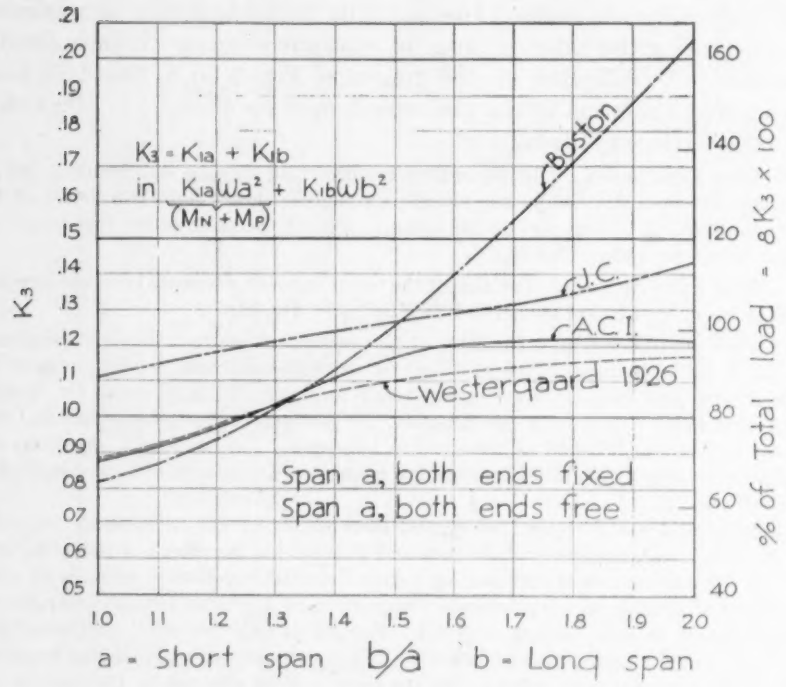
(c) Total Slab Bending. The sum of the equivalent uniform loads causing slab bending in *both* directions ($8K_3w$ or $8K_4w$) for extreme variations in edge restraint are in close agreement with Dr. Westergaard's 1926 formulas. In determining Dr. Westergaard's total slab moment for one direction, the method of averaging simple and continuous panels was followed as described in his paper. For the Joint Committee, this moment was taken as 1½ times the negative moment in accordance with Section (812(b) of that Code.

These criteria are presented as typical illustrations of the consistency by which the ACI regulations properly distribute and provide for the effect of the total load, both on the slab and on the supporting beams. Similar consistency prevails for other conditions of loading and continuity. Comparison of Fig. 7 and 8 demonstrates the wide variation in slab bending caused by changes in edge restraint, and establishes the ACI method of using lines of inflection as a satisfactory measure of true rectangularity. In consequence, it follows that the panel will be affected by the loading and stiffness of adjacent panels in a continuous structure, as prescribed in the Code. Thus, fundamentally, the ACI formulas and method are in substantial accord with accepted theory.

PART 3

General Application

A unique feature of the ACI formula is its flexibility and general applicability to a wide range of different conditions. Through the simple process of modifying the load by the tabulated factors, there results an equivalent uniform load to be used as in one-way construction. Any combination of live and dead load can be treated in this manner with resultant economy of design. Where warranted, accepted methods of adjustment for the transverse torsional resistance of the girders can be included in the determination of the one-way coefficients, as such restraint is in no way peculiar to two-way slabs. In the treatment of unequal panels, the ACI Code surpasses all others in directness and facility, and, through the use of equivalent uniform loads, permits an accurate and easy solution of the complete structure as a rigid frame. Alternately, in simple structures, where arbitrary coefficients are permissible, these may be used as prescribed for one-way spans. Therefore, consistent factors of safety are maintained throughout with other types of floor.



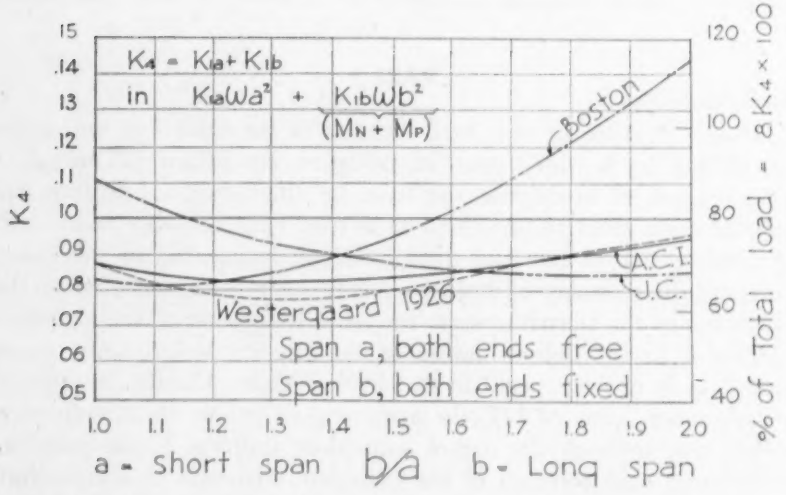


Fig. 7 (top) and 8—Sum of total moment coefficients of slab for two directions

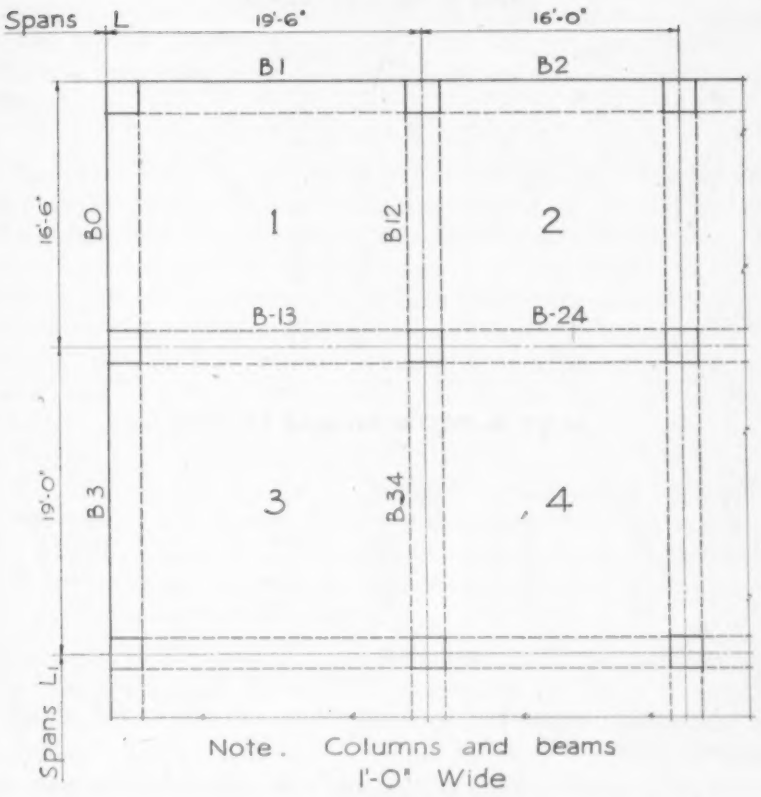


Fig. 9—Typical example

Thus, the ACI Code provides a method of analyzing slabs supported on four sides which is accurate, flexible, and also as now proposed, very simple to apply. An example will be instructive in illustrating the design procedure in a typical building.

Typical example

Let Fig. 9 represent four floor panels at the corner of a building in which the slabs and beams are numbered as shown. First, determine the minimum permissible slab thicknesses in accordance with Sec. 709(b). Using the clear spans, the minimum thickness for Panel 1 becomes

$$\left(\frac{18 + 15}{144}\right) 12 + \left(\frac{18 + 15}{180}\right) 12 = 4.95 \text{ in.}$$

Similarly, in panels 2, 3 and 4, the results are 4.65 in., 4.70 in. and 4.40 in., respectively. A 5-in. slab could therefore be used throughout, provided

TABLE 3—RECTANGULARITY

Panel	r in L Direction $= \frac{mL}{m_1 L_1}$
1	$\frac{.87 \times 18}{.87 \times 15} = 1.20$
2	$\frac{.76 \times 15}{.87 \times 15} = .87$
3	$\frac{.87 \times 18}{.76 \times 18} = 1.15$
4	$\frac{.76 \times 15}{.76 \times 18} = .83$

TABLE 4—TWO-WAY SLAB FACTORS

Panel	r	$X = O$		C	C_1	1—C	1—C_1
		$C_a = C_{b1}$	$C_b = C_{a1}$				
1	1.20	.18	.32	.23	.45	.77	.55
2	.87	.30	.20	.42	.26	.58	.74
3	1.15	.20	.30	.26	.42	.74	.58
4	.83	.32	.18	.45	.23	.55	.77

"C" values taken from Tables 1, 2 or graph Fig .2.

working stresses determined from moments and shears fall within the prescribed values.

As all spans are "at least 2/3 and at most 3/2 of the adjacent continuous span or spans", lines of inflection, when the span under consideration only is loaded, may be determined without recourse to elastic analysis from the ratios m or $m_1 = .87$ for exterior spans and .76 for interior spans. The degree of rectangularity r of the various panels follows directly from Fig. 1, and are given in Table 3. It is only necessary to compute r for one direction in each panel, as with these values factors for determining all moments and shears may be selected from the tables. Generally, only the values of C, $1 - C$. and C_a and C_b for $x = 0$, are required as listed in Table 4. Identical results can be obtained by using $1/r$ with L and L_1 reversed. It is to be noted that for reciprocal values of r with X equal to zero, C_a and C_b are interchanged in amount, and that C_a plus C_b is a constant, in this case .50. This must be true to account for the total panel load. The apparent conformity of panels 3 and 4 with 1 and 2 is coincidental to the shape of the structure.

Given a uniformly distributed total dead and live slab load, w, 100 lb. per sq. ft., an additional interior beam load of 100 lb. per lin. ft., and an exterior wall load of 1000 lb. per lin. ft., shears and bending moments

may be determined as in one-way construction modified by the appropriate factors. As, in the example chosen, the larger of two adjacent spans does not exceed the shorter by more than 20 per cent, the one-way coefficients, prescribed under Section 701(c), may be used in the formulas of 709 (d):

$$V = C_s W \text{ (coefficient)}, \quad M = CWL \ (B.M.C.)$$

The results for slabs may be tabulated as indicated in Table 5: With these moments, minimum slab thicknesses should be checked for structural requirements and reinforcement determined in the usual way. Due regard should be given to the difference in effective depth in the two directions. It will usually be found best practice to place the steel in the direction of the heavier bending moment closest to the surface, The regulations provide that positive reinforcement adjacent to a continuous edge, and for a width not exceeding one-fourth of the shorter dimension of the panel, may be reduced 25 per cent. Accordingly, in this case, bar spacing within a width of 3 ft. 9 in. from B12, B13, B24 and B34, may be increased one third. Negative reinforcement not less than half that required for the unreduced positive moment should be provided across BO, B1, B2 nd B3. The amount of steel at any section is also limited by minimum percentage, maximum bar spacing, and general details as in one-way construction.

Beams may be analyzed in a similar manner with the addition of the effects of special loads. Beam B1 is selected as typical of the method, and shears at intermediate points distance XL from the support will be investigated. From Table 2 the factors listed in Table 6 are found. Using the coefficients prescribed in Section 701(c), the resultant shears and bending moments are as given in Tables 7 and 8 respectively. Where necessary, shear at intermediate sections of the slab may be investigated in a similar manner, using the factors C_s or C_{s1}.

In the design of structures, it is often sufficient to determine the end shears and maximum moments only, as shears at other sections can be estimated with satisfactory accuracy for the spacing of the stirrups. In this instance, it may be convenient to work with equivalent uniformly distributed loads. The factors to be used are $(1—C)$ for bending, and twice the value of C_s when $X = 0$, for shear. The equivalent loads required in the design of all beams of the example are given in Table 9.

With these loads, end shears and bending moments may be calculated in exactly the same manner as in one-way construction. This is of particular advantage in the analysis of rigid frames. Where theoretical refinement is desired, the equivalent loads may be adjusted for the elastic reactions of the slab span normal to the beam by the algebraic addition of the difference in slab end moments divided by its span.

TABLE 5—SLAB SHEARS AND MOMENTS

Panel	Span in Ft. L or L_s	Maximum End Shear in Lbs. per Ft. V or V_l					Maxiumm Positive Moment in Ft. Lbs. per Ft. M or M_l					Maximum Negative Moment in Ft. Lbs. per Ft. M or M_l				
		C_s ×	w ×	L ×	Coef. =	V	C ×	w ×	L^2 ×	B.M.C. =	M	$C^*av.$ ×	w ×	$L^2av.^*$ ×	B.M.C. =	M
1	L = 18	.18	100	18	1.15	372	.23	100	18^2	1/14	530	.325	100	16.5^2	1/10	885
	L_l = 15	.32	100	15	1.15	554	.45	100	15^2	1/14	723	.435	100	16.5^2	1/10	1184
2	L = 15	.30	100	15		450	.42	100	15^2	1/16	590	See Panel 1				885
	L_l = 15	.20	100	15	1.15	345	.26	100	15^2	1/14	419	.245	100	16.5^2	1/10	667
3	L = 18	.20	100	18	1.15	415	.26	100	18^2	1/14	601	.355	100	16.5^2	1/10	965
	L_l = 18	.30	100	18		540	.42	100	18^2	1/16	851	See Panel 1				1198
4	L = 15	.32	100	15		480	.45	100	15^2	1/16	633	See Panel 3				980
	L_l = 18	.18	100	18		324	.23	100	18	1/16	465	See Panel 2				667

*Average values of spans either side of support.

TABLE 6—FACTORS FOR BEAM B1

r \ x	C_b 0.00	.1	.2	.3	.4	1—C
1.20	.32	.29	.24	.18	.10	.77

TABLE 7—SHEARS IN BEAM 1

at X	$\left[\left(C_b \times w \times \frac{L_1}{2}\right) + \text{Beam Load} \times \left(\frac{1}{2} - X\right)\right] \times L \times \text{Coeff.} = \text{Shear } V$							
.0	.32	100	7.5	1000	.5	18′	1.15	15350
.1	.29	"	"	"	.4	"	"	12790
.2	.24	"	"	"	.3	"	"	9900
.3	.18	"	"	"	.2	"	"	6930
.4	.10	"	"	"	.1	"	"	3620

TABLE 8—MOMENTS IN BEAM 1

Max. Moment M	$\left\{\left[(1\text{-}C) \times w \times \frac{L_1}{2}\right] + \text{Beam Load}\right\} \times L^2 \times \text{B.M.C.} = \text{Moment Ft. Lbs.}$						
Max. Pos. Moment..	.77	100	7.5	1000	18^2	1/14	36400
Max. Neg. Moment..	.77	"	"	"	18^2	1/10	51000

For interior beams in Table 9 under Beam load, the first column gives the floor load directly over the beam, and the second column the weight of the beam itself. If preferred, these two terms could be omitted without material error, provided the beam weights are distributed in the unit floor load, and calculations are based on center to center dimensions.

A comparison of the equivalent loads carried by the various spans in this problem illustrates the tendency of the load to be distributed in direct proportion to the stiffness of the respective slab spans. Interior spans are relatively more rigid than end spans, and in wall panels particularly, a greater proportion of the load is attracted to the span in the direction of the smaller moment coefficient. A natural economy of materials is the result.

CONCLUSION

The suggested changes of the 1941 regulations for Slabs Supported on Four Sides are believed to present this subject in the simplest form so far

TABLE 9—BEAM LOADS

Equivalent Uniform Load in Lbs. per Lin. Foot

	For End Shear							For Bending Moment					
Beam	$\left(2 \times C_b \times w \times \frac{L_1}{2}\right) + \left(\text{Beam (1)} + \text{Load (2)}\right)$						= End Shear	$\left((1\text{-}C) \times w \times \frac{L_1}{2}\right) + \left(\text{Beam (1)} + \text{Load (2)}\right)$					= Load for B.M.
B.1	2	.32	100	7.5	1	1000	1480	.77	100	7.5		1000	1578
B.2	2	.20	100	7.5		1000	1300	.58	100	7.5		1000	1435
B13	2 2	.32 .30	100 100	7.5 9.0	100×1	100	1220	.77 .74	100 100	7.5 9.0	100×1	100	1443
B24	2 2	.20 .18	100 100	7.5 9.0	100×1	100	824	.58 .55	100 100	7.5 9.0	100=1	100	1130
BO	2	.18	100	9.0		1000	1324	.55	100	9.0		1000	1495
B.3	2	.20	100	9.0		1000	1360	.58	100	9.0		1000	1522
B12	2 2	.18 .30	100 100	9.0 7.5	100×1	100	974	.55 .74	100 100	9.0 7.5	100×1	100	1250
B34	2 2	.20 .32	100 100	9.0 7.5	100×1	100	1040	.58 .77	100 100	9.0 7.5	100×1	100	1263

(1) Floor load directly over Beam.
(2) Weight of Beam, walls, etc.

devised. Without change in resultant values, notation has been clarified and re-stated with a few familiar symbols. Nonessential formulas and extreneous theory have been eliminated. With these revisions, the Code is properly limited to the statement of the fundamentals requisite for design.

Analysis by this method compares favorably with theoretical standards. By one and the same formula, any variations of live load, rectangularity of panel or inequality of adjacent spans, are handled with equal facility. Computations are short and direct, and provide a clear picture of the equilibrium and elastic actions involved. The many advantages of proportioning two-way slabs from equivalent uniform loads with one-way coefficients are apparent, and the method is made available for convenient use in the suggested new arrangements of this section of the Code.

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7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN (Supplement) November, 1945

Discussion of a paper by Bertin, Di Stasio, and Van Buren:

Slabs Supported on Four Sides*

ERRATA

Part 1, Par. 3
The fifth line should read:
"in *two-way* construction in either direction are obtained."

Fig. 2
Curve marked "1-C (read down)" should also be marked below the curve:
"C (read up)"

Fig. 7
The second line of note near bottom of diagram should read: "span *b* both ends free" instead of "span *a* both ends free."

Table 9
Sub-heading ending "-End Shear", should read:
"- Load for End Shear."

Table 5
Max. Pos Moment under L^2, last line "18," should read: "18²".

By WILLIAM C. SPIKER†

Well planned, clearly stated and referring to a very important subject, this paper should receive a great deal of attention.

The writer has checked the moments and loads per foot on beams and slabs reported in Tables 9 and 5. His figures agree almost exactly in nearly all cases with those in the tables assuming that the beams do not deflect.

The basis of the writers method is as follows:

The distribution of any load on any point on any slab is inversely in proportion to the total deflection of that point under any load calculated along the various routes which pick up some of the load as it travels to the columns. The distribution of the total uniform load on any panel is directly proportional to the distribution of the load on the one square foot of the panel at the point of maximum deflection in the panel, including the deflection of the beams if there are any.

*ACI JOURNAL, June 1945; *Proceedings* V. 41, p. 537.
†Consulting Engineer, Atlanta, Ga.

Practically the only problem is to calculate the maximum deflection. If the beams do not deflect and θ at the slab supports is the same for both ends of both slab strips which contain the point of maximum deflection the distribution is inversely as the fourth power of the two spans at right angles to each other. However, the writer recommends the deflection formulas contained in "Statically Indeterminate Stresses" by Parcell and Maney and that a complete tabulation of coefficients and distances "x"—distances from supports to points of inflection—be made if one has a great deal of calculating of two way slabs to do.

Current ACI Standards

Standards of the American Concrete Institute adopted since the inauguration of the current procedures for their consideration and promulgation under the supervision of the Standards Committee (see ACI Directory, 1944, p. 131) are published in the pages which follow—559-704, collected in one publication. Each Standard will also be available in separate prints as heretofore. New editions of the collected ACI Standards will be issued as rapidly as is justified by the completion of technical committee work. Some of the present Standards have had some few editorial revisions. They include changes in the substance of the texts as approved by the ACI Conventions which adopted them and as subsequently ratified by letter ballot of the ACI Membership. Not included here are "proposed standards" presented in recent years, nor proposed or ratified Standards prior to 1937. Some of the latter will have thorough review and eventually come before the Institute for consideration.

ACI STANDARDS—1945

	Pages
Building Regulations for Reinforced Concrete (ACI 318-41)	559-620
Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)	621-624
Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)	625-650
Recommended Practice for the Design of Concrete Mixes (ACI 613-44)	651-672
Specifications for Concrete Pavements and Bases (ACI 617-44)	673-700
Specification for Cast Stone (ACI 704-44)	701-704

[To facilitate selective distribution, **separate prints** of this ACI Standard (title 41-23) are currently available in covers from ACI at 50 cents each—quantity quotations on request.]

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ACI Standard

Building Regulations for Reinforced Concrete (ACI 318-41)*

Reported by ACI Committee 318

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CHAPTER 1—GENERAL

101—Scope

(*a*) These regulations cover the use of reinforced concrete and plain concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of these materials. In all matters pertaining to design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102—Permits and drawings

(*a*) Drawings and typical details of all reinforced concrete construction showing the size and position of all structural members, metal reinforcement, design strength of concrete, and the live load used in the design shall be filed with the building department as a permanent record before a permit to construct such work will be

*Adopted as a Standard of the American Concrete Institute at its 37th Annual Convention, February 20, 1941 as reported by Committee 318; Ratified by Letter Ballot July 21, 1941 (with editorial corrections from previous printings, in accordance with "Errata" leaflet issued 1943.) The Committee acknowledges the active cooperation of the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute.

†Deceased (succeeded 1944 by Arthur J. Boase as chairman). Names are those representing committee personnel when report was presented.

issued. All plans submitted for approval or use on the work shall clearly show the strength of concrete at a specified age for which all parts of the structure were designed. Calculations pertaining to the design shall be filed with the drawings when required by the Commissioner of Buildings.

103—Special systems of reinforced concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by these regulations shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

104—Definitions

(*a*) The following terms are defined for use in this code:

Aggregate—Inert material which is mixed with portland cement and water to produce concrete.

Column—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip—A portion of a flat slab panel one-half panel in width consisting of the two adjacent quarter-panels on either side of the column center lines and extending through the panel in the direction of the span considered for bending.

Combination Column—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column—A column in which a steel or cast-iron section is completely encased in concrete containing spiral and longitudinal reinforcement.

Concrete—A mixture of portland cement, fine aggregate, coarse aggregate and water.

Deformed Bar—Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling. Wire mesh with welded intersections not farther apart than twelve

inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 W. & M. gage may be rated as a deformed bar.

Diagonal Band—A group of reinforcing bars covering a width approximately 0.4 the average span, placed symmetrically with respect to the diagonal running from corner to corner of the panel of a flat slab.

Direct Band—A group of reinforcing bars, covering a width approximately 0.4 of l_1, placed symmetrically with respect to the center lines of the supporting columns of a flat slab.

Drop Panel—The structural portion of a flat slab which is thickened in the area surrounding the column capital.

Effective Area of Concrete—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the flexural member.

Effective Area of Reinforcement—The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Flat Slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting columns.

Middle Strip—A portion of a flat slab panel one-half panel in width, symmetrical about the panel center line and extending through the panel in the direction of the span considered for bending.

Paneled Ceiling—A flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length—The distance along a panel side from center to center of columns of a flat slab.

Pedestal—An upright compression member whose height does not exceed three times its least lateral dimension.

Plain Concrete—Concrete without reinforcement, or reinforced only for shrinkage or temperature changes.

Ratio of Reinforcement—The ratio of the effective area of the reinforcement to the effective area of the concrete at any section of a flexural member.

Reinforced Concrete—Concrete in which reinforcement other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

Surface Water—The water carried by the aggregate except that held by absorption within the aggregate particles themselves.

CHAPTER 2—MATERIALS AND TESTS

200—Notation

D = Deflection of a floor member under load test.

L = Span of member under load test.

t = The total thickness or depth of a member under load test.

201—Tests

(*a*) The Commissioner of Buildings, or his authorized representative, shall have the right to order the test of any material entering into concrete or reinforced concrete when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(*b*) Tests of materials and of concrete shall be made in accordance with the requirements of the American Society for Testing Materials as noted elsewhere in this chapter. The complete records of such tests shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work, and shall be preserved by the engineer or architect for two years after the completion of the structure.

202—Load tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given in the following:

$$D = \frac{.001\,L^2}{12\,t} \qquad (1)$$

all terms expressed in the same units.

If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the member or portion of the structure shows a recovery of at least seventy-five per cent of the observed deflection.

203—Supervision

(*a*) All concrete work shall be supervised by the architect or engineer responsible for its design, or by a competent representative responsible to the architect or engineer. A record shall be kept of such supervision, which record shall cover the quality and quantity of concrete materials, the mixing and placing of the concrete, and the placing of the reinforcing steel. A complete record shall also be kept of the progress of the work and of the temperatures, when these fall below 40 degrees F., and of the protection given to the concrete while curing. This record shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work and shall be preserved by the architect or engineer for two years after the completion of the work.

204—Portland cement

(*a*) Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Serial Designation: C9-38) or the "Standard Specifications for High-Early-Strength Portland Cement" (A.S.T.M. Serial Designation: C74-39).

205—Concrete aggregates

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (A. S. T. M. Serial Designation: C33-40), provided however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness, fire-resistance, and wearing qualities may be used under Section 302(a) Method 2, where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between sides of the forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars.

206—Water

(*a*) Water used in mixing concrete shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials, or other deleterious substances.

207—Metal reinforcement

(*a*) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A15-39), or for "Rail-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A16-35), or for "Axle-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A160-39).

(*b*) Cold-drawn wire or welded wire fabric for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (A. S. T. M. Serial Designation: A82-34), or "Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement" (A. S. T. M. Serial Designation: A185-37).

(*c*) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Bridges and Buildings" (A. S. T. M. Serial Designation: A7-39).

(*d*) Cast-iron sections for composite columns shall conform to the "Tentative Specifications for Cast Iron Pit-cast Pipe for Water and Other Liquids" (A.S.T.M. Serial Designation: A44-39T).

208—Storage of materials

(*a*) Cement and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used for concrete.

CHAPTER 3—CONCRETE QUALITY AND WORKING STRESSES

300—Notation

f_c = Compressive unit stress in extreme fiber of concrete in flexure.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

f_r = Allowable unit stress in the metal core of a composite column.

f_s = Tensile unit stress in longitudinal reinforcement; nominal working stress in vertical column reinforcement.

f_v = Tensile unit stress in web reinforcement.

n = Ratio of modulus of elasticity of steel to that of concrete.

u = Bond stress per unit of surface area of bar.

v = Shearing unit stress.

v_c = Shearing unit stress permitted on the concrete.

301—Concrete quality

(*a*) For the design of reinforced concrete structures, the value of f'_c used for determining the working stresses as stipulated in Section 305 shall be based on the specified minimum ultimate 28-day compressive strength of the concrete, or on the specified minimum ultimate compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) All concrete exposed to the action of the weather shall have a water-content of not to exceed six gallons per sack of cement.*

302—Determination of strength-quality of materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

*In climates where frost action is not severe this section should be omitted.

TABLE 302(a)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Content U. S. Gallons Per 94-lb. Sack of Cement	Assumed Compressive Strength at 28 Days—p.s.i.
7½	2000
6¾	2500
6	3000
5	3750

NOTE—In interpreting this table, surface water carried by the aggregate must be included as part of the mixing water in computing the water-content.

Method 2—Controlled Concrete:

Water-content other than shown in Table 302(*a*) may be used provided that the strength-quality of the concrete proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A.S.T.M. Serial Designation: C39-39). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The maximum allowable water-content for the concrete for the structure shall be as determined from this curve and shall correspond to a strength which is fifteen per cent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

303—Tests on concrete

(*a*) The Commissioner of Buildings shall require a reasonable number of compression tests to be made during the progress of the work. Such tests shall be made in accordance with the "Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field" (A. S. T. M. Serial Designation C31-39), and cured in accordance with the requirements for laboratory control tests.

(*b*) Not less than three specimens shall be made for each test; nor less than one test for each 250 cu. yd. of concrete.

(*c*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths of the concrete is established by test for the materials and proportions used.

(*d*) Where the average strength of the laboratory control cylinders for any portion of the structure falls below the minimum ultimate compressive strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mixture or in the water-content for the remaining portion of the structure. In cases where the average strength of the cylinders cured on the job falls be-

low the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength. If the average strength of either the laboratory control cylinders or the cylinders cured on the job falls below the required strength, load tests as specified in Section 202 may be required on the portion of the structure so affected.

304—Concrete proportions and consistency

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than thirty per cent nor greater than fifty per cent of the total, except that these proportions do not necessarily apply to light-weight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for ready mixed concrete shall conform to the "Standard Specifications for Ready-Mixed Concrete" (A. S. T. M. Serial Designation: C94-38).

305—Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(a) where f'_c equals the minimum specified ultimate compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

306—Allowable unit stresses in reinforcement

Unless otherwise provided in these Regulations, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)

and (f_s = Tensile unit stress in web reinforcement)

20,000 p.s.i. for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

*Wherever practicable such measurement shall be by weight rather than by volume.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable Unit Stresses				
		For Any Strength of Concrete as Fixed by Test in Accordance with Section 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302			
			f'_c = 2000 p.s.i. n = 15	f'_c = 2500 p.s.i. n = 12	f'_c = 3000 p.s.i. n = 10	f'_c = 3750 p.s.i. n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c	$0.03f'_c$ but not to exceed 75 p.s.i.	60	75	75	75
‡Bond: u						
In beams and slabs and one-way footings:						
Plain bars	u	$0.04f'_c$ but not to exceed 160 p.s.i.	80	100	120	150
Deformed bars	u	$0.05f'_c$ but not to exceed 200 p.s.i.	100	125	150	188
In two-way footings:						
Plain bars (hooked)	u	$0.045f'_c$ but not to exceed 160 p.s.i.	90	113	135	160
Deformed bars (hooked)	u	$0.056f'_c$ but not to exceed 200 p.s.i.	112	140	168	200
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area or less†	f_c	$0.375f'_c$	750	938	1125	1405

*See Section 807. **See Section 905(*a*) and 808(*a*).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Section 903(*a*)), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 p.s.i. for plain bars and 250 p.s.i. for deformed bars. The values given for two-way footings include an allowance for special anchorage.

18,000 p.s.i. for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

(*b*) *Tension in One-Way Slabs of Not More Than 12 Feet Span* (f_s = Tensile unit stress in main reinforcement).

For the main reinforcement, $\frac{3}{8}$ inch or less in diameter, in one-way slabs, 50 per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(*c*) *Compression, Vertical Column Reinforcement*

(f_s = Nominal working stress in vertical column reinforcement).

Forty per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(f_r = Allowable unit stress in the metal core of composite and combination columns):

Structural steel sections	16,000 p.s.i.
Cast iron sections	10,000 p.s.i.
Steel pipe	See limitations of Section 1106(b)

(*d*) *Compression, Flexural Members*

For compression reinforcement in flexural members see Section 706(b).

CHAPTER 4—MIXING AND PLACING CONCRETE

401—Preparation of equipment and place of deposit

(*a*) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the spaces to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and masonry filler units that will be in contact with concrete shall be well drenched (except in freezing weather), and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(*b*) Water shall be removed from place of deposit before concrete is placed unless otherwise permitted by the Commissioner of Buildings.

402—Mixing of concrete

(*a*) Unless otherwise authorized by the Commissioner of Buildings, the mixing of concrete shall be done in a batch mixer of approved type.

(*b*) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(*c*) For job mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer. A longer mixing period may be required for mixers larger than one cubic yard capacity.

(*d*) Ready-mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the "Standard Specifications for Ready-Mixed Concrete" (A. S. T. M. Serial Designation C94-38).

403—Conveying

(*a*) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

(*b*) Equipment for chuting, pumping and pneumatically conveying concrete shall be of such size and design as to insure a practically continuous flow of concrete at the delivery end without separation of the materials.

404—Depositing

(*a*) Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. The

concreting shall be carried on at such a rate that the concrete is at all times plastic and flows readily into the spaces between the bars. No concrete that has partially hardened or been contaminated by foreign materials shall be deposited on the work, nor shall retempered concrete be used.

(*b*) When concreting is once started, it shall be carried on as a continuous operation until the placing of the panel or section is completed. The top surface shall be generally level. When construction joints are necessary, they shall be made in accordance with Section 508.

(*c*) All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around the reinforcement and embedded fixtures and into the corners of the forms. Vibrators may be used to aid in the placement of the concrete provided they are used under experienced supervision, and the forms are designed to withstand their action.

(*d*) Where conditions make compacting difficult, or where the reinforcement is congested, batches of mortar containing the same proportions of cement to sand as used in the concrete, shall first be deposited in the forms to a depth of at least one inch.

405—Curing

(*a*) In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days.

406—Cold weather requirements

(*a*) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(*b*) All concrete materials and all reinforcement, forms, fillers and ground with which the concrete is to come in contact, shall be free from frost. Whenever the temperature of the surrounding air is below 40 degrees Fahrenheit, all concrete when placed in the forms shall have a temperature of between 60 and 90 degrees Fahrenheit and shall be maintained at a temperature of not less than 50 degrees Fahrenheit for at least 72 hours for normal concrete or 24 hours for high-early-strength concrete, or for as much more time as is

necessary to insure proper rate of curing of the concrete. The housing, covering or other protection used in connection with curing shall remain in place and intact at least twenty-four hours after the artificial heating is discontinued. No dependence shall be placed on salt or other chemicals for the prevention of freezing. Manure, when used for protection, shall not be allowed to come into contact with the concrete.

CHAPTER 5—FORMS AND DETAILS OF CONSTRUCTION

501—Design of forms

(*a*) Forms shall conform to the shape, lines, and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

502—Removal of forms

(*a*) Forms shall be removed in such manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, the removable floor forms, beam and girder sides, column and similar vertical forms may be removed after twenty-four hours, providing the concrete is sufficiently hard not to be injured thereby. In no case shall the supporting forms or shoring be removed until the members have acquired sufficient strength to support safely their weight and the load thereon. The results of suitable control tests may be used as evidence that the concrete has attained such sufficient strength.

503—Pipes, conduits, etc., embedded in concrete

(*a*) Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete except in passing through from one side to the other of a floor, wall or beam. Electric conduits and other pipes whose embedment is allowed shall not, with their fittings, displace that concrete of a column on which stress is calculated or which is required for fire protection, to greater extent than four per cent of the area of the cross section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced concrete, provided they are not exposed to rusting or other deterioration, are of uncoated iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over two inches, and are spaced not less than three diameters on centers. Embedded pipes

or conduits other than those merely passing through, shall not be larger in outside diameter than one-third the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than three diameters on centers, nor so located as unduly to impair the strength of the construction. Circular uncoated or galvanized electric conduit of iron or steel may be considered as replacing the displaced concrete.

504—Cleaning and bending reinforcement

(*a*) Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for stirrups and ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. Bends for other bars shall be made around a pin having a diameter not less than six times the minimum thickness of the bar, except that for bars larger than one inch, the pin shall be not less than eight times the minimum thickness of the bar. All bars shall be bent cold.

505—Placing reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs and spacers. The minimum clear distance between parallel bars shall be one and one-half times the diameter for round bars and twice the side dimension for square bars. If special anchorage as required in Section 903 is provided, the minimum clear distance between parallel bars shall be equal to the diameter for round bars and one and one-half times the side dimension for square bars. In no case shall the clear distance between bars be less than one in., nor less than one and one-third times the maximum size of the coarse aggregate.

(*b*) When wire or other reinforcement, not exceeding one-fourth inch in diameter is used as reinforcement for slabs not exceeding ten feet in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at mid-span; provided such reinforcement is either continuous over, or securely anchored to the support.

506—Splices and offsets in reinforcement

(*a*) In slabs, beams and girders, splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the minimum spacing of bars shall be as specified in Section 505.

(*b*) Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion shall not be more than 1 in 6, and in the case of tied columns the ties shall be spaced not over three inches on centers for a distance of one foot below the actual point of offset.

507—Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than $\frac{5}{8}$ inch in diameter and one and one-half inches for bars $\frac{5}{8}$ inch or less in diameter.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of metal reinforcement shall be at least three-fourths inch.

(*c*) If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

508—Construction joints

(*a*) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a joint is to be made, the surface of the concrete shall be thoroughly cleaned and all laitance removed. In addition to the foregoing, vertical joints shall be thoroughly wetted but not saturated, and slushed with a coat of neat cement grout immediately before placing of new concrete.

(*b*) At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(*c*) Construction joints in floors shall be located near the middle of the spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement

CHAPTER 6—DESIGN—GENERAL CONSIDERATIONS

600—Notation

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

n = Ratio of modulus of elasticity of steel to that of concrete = $\frac{E_s}{E_c}$; assumed as equal to $\frac{30{,}000}{f'_c}$.

601—Assumptions

(*a*) The design of reinforced concrete members shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending. The following assumptions shall be made:

1. The steel takes all the tensile stress.
2. In determining the ratio n for design purposes, the modulus of elasticity for the concrete shall be assumed as $1000\,f'_c$, and that for steel as 30,000,000 p.s.i.

602—Design loads

(*a*) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

603—Resistance to wind forces

(*a*) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(*b*) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead- and live-loads.

(*c*) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the allowable stresses for combined live- and dead-loads provided in Sections 305, 306 and 1110 by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

CHAPTER 7—FLEXURAL COMPUTATIONS

700—Notation

A = Span length between opposite supports in one direction.

B = Span length at right angles to A.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the diameter of a round bar or side of a square bar.

e_A = Factor modifying r_A, used in obtaining an equivalent uniform load for bending moments on span A.

e_B = Factor modifying r_B, used in obtaining an equivalent uniform load for bending moments on span B.

E = The modulus of elasticity of concrete in compression.

F_AA = The distance between lines of inflection in span A, considering span A only to be loaded.

F_BB = The distance between lines of inflection in span B, considering span B only to be loaded.

F_A = Ratio of the distance between assumed inflection points of the span A to span A in an isolated strip extending the entire width of the structure when a uniformly distributed load is applied to span A only.

F_B = Ratio as defined above, but applying to span B.

h = Unsupported length of a column.

I = Moment of inertia of a section about the neutral axis for bending.

K = The stiffness factor, that is, the moment of inertia divided by the span.

K_A = Stiffness factor $\frac{I}{A}$ for span A of panel AB.

K_B = Stiffness factor $\frac{I}{B}$ for span B of panel AB.

K_{AR} = Stiffness factors for any span adjacent to and continuous with span A.

K_{BR} = Stiffness factors for any span adjacent to and continuous with span B.

l = Span length of slab or beam.

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment (See Section 701).

N = The sum of the lengths of those edges of panel AB which are also edges of adjacent panels continuous with AB.

$q_A = 6r_A\ (1 - e_A)$.

$q_B = 6r_B\ (1 - e_B)$.

r_A = Proportion of the total load carried by span A of slab.

r_B = Proportion of the total load carried by span B of slab.

t_1 = Minimum total thickness of slab.

w = Uniformly distributed load per unit of length of beam or per unit area of slab.

x = Distance from face of support to point in span.

701—General requirements

(*a*) All members of frames or continuous construction shall be designed to resist at all sections the maximum moments and shears produced by dead load, live load and wind load, as determined by the theory of elastic frames in which the simplified assumptions of Section 702 may be used.

(*b*) Approximate methods of frame analysis are satisfactory for buildings of usual types of construction, spans and story heights.

(*c*) In the case of two or more approximately equal spans (the larger of two adjacent spans not exceeding the shorter by more than 20 per cent) with loads uniformly distributed, where the unit live load does not exceed three times the unit dead load, design for the following moments and shears is satisfactory:

Positive moment at center of span

End spans ..... $\frac{1}{14}\ wl'^2$

Interior spans ..... $\frac{1}{16}\ wl'^2$

Negative moment at exterior face of first interior support

Two spans ..... $\frac{1}{9}\ wl'^2$

More than two spans ..... $\frac{1}{10}\ wl'^2$

Negative moment at other faces of interior supports	$\frac{1}{11} wl'^2$
Negative moment at face of all supports for, (a) slabs with spans not exceeding ten feet, and (b) beams and girders where ratio of sum of column stiffnesses to beam stiffness exceeds eight	$\frac{1}{12} wl'^2$
Shear in end members at first interior support	$1.15 \frac{wl'}{2}$
Shear at other supports	$\frac{wl'}{2}$

702—Conditions of design*

(*a*) *Arrangement of Live Load*

1. The live load may be considered to be applied only to the floor under consideration, and the far ends of the columns may be assumed as fixed.

2. Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans.

(*b*) *Span length*

1. The span length, l, of members that are not built integrally with their supports shall be the clear span plus the depth of the slab or beam but shall not exceed the distance between centers of supports.

2. In analysis of continuous frames, center to center distances, l and h, may be used in the determination of moments. Moments at faces of supports may be used for design of beams and girders.

3. Solid or ribbed slabs with clear spans of not more than ten feet that are built integrally with their supports may be designed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and the width of beams otherwise neglected.

(*c*) *Stiffness*

1. The stiffness, K, of a member is defined as EI divided by l or h.

2. In computing the value of I of slabs, beams, girders, and columns, the reinforcement may be neglected. In T-shaped sections allowance shall be made for the effect of flange.

*Chapter 7 deals with floor members only. For moments in columns see Section 1108.

3. Any reasonable assumption may be adopted as to relative stiffness of columns and of floor system. The assumption made shall be consistent throughout the analysis.

(*d*) *Haunched Floor Members*

1. When members are widened near the supports, the additional width may be neglected in computing moments, but may be considered as resisting the resulting moments and shears.

2. When members are deepened near the supports, they may be analyzed as members of constant depth provided the minimum depth only is considered as resisting the resulting moments; otherwise an analysis taking into account the variation in depth is required. In any case, the actual depth may be considered as resisting shear.

(*e*) *Limitations*

1. Wherever at any section positive reinforcement is indicated by analysis, the amount provided shall be not less than .005 $b'd$ except in slabs of uniform thickness.

2. Not less than 0.005 $b'd$ of negative reinforcement shall be provided at the outer end of all members built integrally with their supports.

3. Where analysis indicates negative reinforcement along the full length of a span, the reinforcement need not be extended beyond the point where the required amount is 0.0025 $b'd$ or less.

4. In slabs of uniform thickness the minimum amount of reinforcement in the direction of the span shall be:

For structural, intermediate and hard grades and rail steel . . . . . 0.0025 bd

For steel having a minimum yield point of 56,000 p. s. i. . . . . 0.002 bd

703—Depth of beam or slab

(*a*) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face of the structural members. Any floor finish not placed monolithically with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithically with the structural slab in buildings of the warehouse or industrial class, there shall be placed an additional depth of one-half inch over that required by the design of the member.

704—Distance between lateral supports

(*a*) The clear distance between lateral supports of a beam shall not exceed thirty-two times the least width of compression flange.

705—Requirements for T-beams

(*a*) In T-beam construction the slab and beam shall be built integrally or otherwise effectively bonded together. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(*b*) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(*c*) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a joist in concrete joist floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, nor in any case eighteen inches.

(*d*) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Section 505 relating to the spacing of bars, and 404(d), relating to the placing of concrete shall be fully met.

(*e*) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(*f*) Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

706—Compression steel in flexural members

(*a*) Compression steel in beams, girders, or slabs shall be anchored by ties or stirrups not less than ¼ inch in diameter spaced not farther apart than 16 bar diameters, or 48 tie diameters. Such stirrups or ties shall be used throughout the distance where the compression steel is required.

(*b*) The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from the calculations assuming a straight-line relation between stress and strain and the modular ratio given in Section 601, but not of greater value than the allowable stress in tension.

707—Shrinkage and temperature reinforcement

(*a*) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area *bd*, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than eighteen inches:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0022

708—Concrete joist floor construction

(*a*) Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than thirty inches face to face. The ribs shall be straight, not less than four inches wide, nor of a depth more than three times the width.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the joists are used, and the fillers are so placed that the joints in alternate rows are staggered, the vertical shells of the fillers in contact with the joists may be included in the calculations involving shear or negative bending moment. No other portion of the fillers may be included in the design calculations.

(*c*) The concrete slab over the fillers shall be not less than one and one-half inches in thickness, nor less in thickness than one-twelfth of the clear distance between joists. Shrinkage reinforcement in the slab shall be provided as required in Section 707.

(*d*) Where removable forms or fillers not complying with (*b*) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between joists and in no case less than two inches. Such slab shall be reinforced at right angles to the joists with a minimum of .049 sq. in. of reinforcing steel per foot of width, and in slabs on which the prescribed live load does not exceed fifty lb. per sq. ft., no additional reinforcement shall be required.

(*e*) When the finish used as a wearing surface is placed monolithically with the structural slab in buildings of the warehouse or industrial class, the thickness of the concrete over the fillers shall be one-half inch greater than the thickness used for design purposes.

(*f*) Where the slab contains conduits or pipes, the thickness shall not be less than one inch plus the total over-all depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to impair the strength of the construction.

709—Floors with supports on four sides (1) (2)

(*a*) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with burned clay or concrete tile fillers, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The supports for the floor slabs may be walls, reinforced concrete beams, or steel beams fully encased in concrete.

Footnotes:

(1) For comparative use the moment of inertia of a slab shall be taken as that of the total plain concrete section.

(2) Formulas for F_A, F_B, e_A, e_B, r_A, r_B. (See "Slabs Supported on Four Sides" by J. DiStasio and M. P. van Buren, JOURNAL of the A. C. I., January-February, 1936).

End Span, continuous at one end only

$$F_A = 1 - \frac{0.25}{1 + \frac{7K_A}{8K_{AB}}} \qquad (7)$$

Interior continuous span with K_{AB} the same for both adjacent spans continuous with A

$$F_A = \sqrt{1 - \frac{1}{1.5 + \frac{7K_A}{8K_{AB}}}} \qquad (8)$$

For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AB} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AB} for the span at the other end.

To obtain F_B replace K_A with K_B and K_{AB} with K_{BB}.

$$r_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)^3} = 1 - r_B \qquad (9)$$

$$e_A = \frac{2}{4 - \frac{F_B B}{F_A A}} \qquad (10a) \qquad e_B = \frac{2}{4 - \frac{F_A A}{F_B B}} \qquad (10b)$$

$e_A = 1.0$ for $\frac{F_B B}{F_A A} = 2$ $\qquad$ $e_B = 0.5$, as $\frac{F_A A}{F_B B} = 0$

The total load carried by a strip of slab of unit width, span A, equals $r_A w A$ and is considered to vary in intensity from $r_A w\ (3e_A - 2)$ at the center of the span, to $r_A w\ (4 - 3\ e_A)$ at the supports.

The total load carried by a beam of span A, one-half panel tributary width, equals

$$(1 - r_A)\frac{wBA}{2} \qquad (11)$$

and varies uniformly in intensity from $(1 + 2r_A - 3e_A r_A)\ \frac{wB}{2}$ at the center of the span to

$(1 - 4r_A + 3e_A r_A)\ \frac{wB}{2}$ at the supports.

When considering the B spans use the above expressions, replacing A with B, B with A, r_A with r_B, and e_A with e_B.

(*Footnote (2) continued next page*)

(*b*) *Minimum Slab Thickness*

The slab thickness shall satisfy prescribed working stresses and shall be not less than

$$t_1 = \frac{A + B - 0.10N}{72} \quad \ldots\ldots\ldots\

$$\frac{B}{2}(1 - e_A r_A)w \quad \ldots\ldots (4)$$

The shear at any section at a distance x from the face of the support shall be taken as:

Slab: Strip of unit width, span A,

$$\left(\frac{A}{2} - x\right)\left(r_A - \frac{q_A x}{A}\right)w \quad \ldots\ldots (5)$$

Beam: Span A, carrying one half of load from panel, width B,

$$\frac{B}{2}\left(\frac{A}{2} - x\right)\left(1 - r_A + \frac{q_A x}{A}\right)w \quad \ldots\ldots (6)$$

For span B, use the above expressions substituting A for B, B for A, e_B for e_A, r_A for r_B, and q_B for q_A.

The factors e_A, r_A, etc., may be taken from Table 2, footnote (2) below after the ratio $F_A A/F_B B$ or $F_B B/F_A A$ on which they depend has been determined by the aid of Table 1, footnote (2); or the several factors may be computed from the formulas which appear in the footnote (2).

(*d*) *Arrangement of Reinforcement*

1. In any panel, the reinforcement per unit width in the long direction shall be at least one-third of that provided in the short direction.

2. The positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced twenty-five per cent.

3. At a non-continuous edge negative moment reinforcement per unit width in amount at least as great as one-half of that required for maximum positive moment for the center one-half of the panel shall be provided across the entire width of the exterior support.

4. The spacing of the reinforcement shall be not more than three times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

710—Maximum spacing of principal slab reinforcement

(*a*) In slabs other than concrete joist floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 707(a).

CHAPTER 8—SHEAR AND DIAGONAL TENSION

800—Notation

A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

α = Angle between inclined web bars and axis of beam.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

f_v = Tensile unit stress in web reinforcement.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

t_2 = Thickness of flat slab without drop panels, or the thickness of flat slab through the drop panels where such are used.

t_3 = Thickness of flat slab (with drop panels) at points outside the drop panel.

v = Shearing unit stress.

V = Total shear.

V' = Excess of the total shear over that permitted on the concrete.

801—Shearing unit stress

(*a*) The shearing unit stress v, as a measure of diagonal tension, in reinforced concrete flexural members shall be computed by formula (12):

$$v = \frac{V}{bjd} \qquad (12)$$

(*b*) For beams of I or T section, b' shall be substituted for b in formula (12).

(*c*) In concrete joist floor construction, where burned clay or concrete tile are used, b' may be taken as a width equal to the thickness of the concrete web plus the thicknesses of the vertical shells of the concrete or burned clay tile in contact with the joist as in Section 708(b).

(*d*) When the value of the shearing unit stress computed by formula (12) exceeds the shearing unit stress v_e permitted on the concrete of an unreinforced web (see Section 305), web reinforcement shall be provided to carry the excess.

802—Types of web reinforcement

(*a*) Web reinforcement may consist of:

1. Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

2. Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30 degrees or more thereto.

3. Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.

4. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the concrete by the reinforcement (See Section 804(f)).

(*b*) Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Section 904.

803—Stirrups

(*a*) The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by formula (13).

$$A_v = \frac{V's}{f_v\, jd} \qquad (13)$$

(*b*) Inclined stirrups shall be proportioned by formula (15) (Section 804(d).)

(*c*) Stirrups placed perpendicular to the longitudinal reinforcement shall not be used alone as web reinforcement when the shearing unit stress (v) exceeds $0.08f'_c$.

804—Bent bars

(*a*) When the web reinforcement consists of a single bent bar or of a single group of bent bars the required area of such bars shall be computed by formula (14).

$$A_v = \frac{V'}{f_v \sin \alpha} \qquad (14)$$

(*b*) In formula (14) V' shall not exceed $0.040\, f'_c\, bjd$.

(*c*) Only the center three-fourths of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

(*d*) Where there is a series of parallel bent bars, the required area shall be determined by formula (15).

$$A_v = \frac{V's}{f_v jd\ (sin\ \alpha + cos\ \alpha)} \qquad (15)$$

(*e*) When bent bars, having a radius of bend of not more than two times the diameter of the bar are used alone as web reinforcement, the allowable shearing unit stress shall not exceed $0.060\ f'_c$. This shearing unit stress may be increased at the rate of $0.01\ f'_c$ for each increase of four bar diameters in the radius of bend until the maximum allowable shearing unit stress is reached. (See Section 305(a).)

(*f*) The shearing unit stress permitted when special arrangements of bars are employed shall be that determined by making comparative tests, to destruction, of specimens of the proposed system and of similar specimens reinforced in conformity with the provisions of this code, the same factor of safety being applied in both cases.

805—Combined web reinforcement

(*a*) Where more than one type of reinforcement is used to reinforce the same portion of the web, the total shearing resistance of this portion of the web shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once, and no one type of reinforcement shall be assumed to resist more than $\frac{2\ V'}{3}$.

806—Spacing of web reinforcement

(*a*) Where web reinforcement is required it shall be so spaced that every 45 degree line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement. If a shearing unit stress in excess of $0.06\ f'_c$ is used, every such line shall be crossed by at least two such lines of web reinforcement.

807—Shearing stress in flat slabs

(*a*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_2 - 1\frac{1}{2}$ in. beyond the edge of the column capital

and parallel or concentric with it, shall not exceed the following values when computed by formula (12) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.):

1. $0.03\,f'_c$, when at least 50 per cent of the total negative reinforcement in the column strip passes directly over the column capital.

2. $0.025\,f'_c$, when 25 per cent or less of the total negative reinforcement in the column strip passes directly over the column capital.

3. For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(*b*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_3 - 1\frac{1}{2}$ in. beyond the edge of the drop panel and parallel with it shall not exceed $0.03\,f'_c$ when computed by formula (12) (in which d shall be taken as $t_3 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in the column strip must be within the width of strip directly above the drop panel.

808—Shear and diagonal tension in footings

(*a*) In isolated footings the shearing unit stress computed by formula (12) on the critical section (see 1205(a)), shall not exceed $0.03\,f'_c$, nor in any case shall it exceed 75 p.s.i.

CHAPTER 9—BOND AND ANCHORAGE

900—Notation

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

Σo = Sum of perimeters of bars in one set.

u = Bond stress per unit of surface area of bar.

V = Total shear.

901—Computation of bond stress in beams

(*a*) In flexural members in which the tensile reinforcement is parallel to the compression face, the bond stress at any cross section shall be computed by formula (16).

$$u = \frac{V}{\Sigma o\, jd} \qquad (16)$$

in which V is the shear at that section.

(*b*) Adequate end anchorage shall be provided for the tensile reinforcement in all flexural members to which formula (16) does not apply, such as footings, brackets and other tapered or stepped beams in which the tensile reinforcement is not parallel to the compression face.

902—Ordinary anchorage requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar shall be extended at least twelve diameters beyond the point at which it is no longer needed to resist stress. In cases where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond, the bar shall extend into a region of compression and be anchored by means of a standard hook or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the

bar and either made continuous with the positive reinforcement or anchored in a region of compression.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of ten or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks, or other adequate anchorage.

(*c*) In simple beams, or at the outer or freely supported ends of end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of twelve or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

903—Special anchorage requirements

(*a*) Where increased shearing or bond stresses are permitted because of the use of special anchorage (See Section 305), every bar shall be terminated in a standard hook in a region of compression, or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the negative or positive reinforcement.

904—Anchorage of web reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

1. Welding to longitudinal reinforcement.

2. Hooking tightly around the longitudinal reinforcement through 180 degrees.

3. Embedment above or below the mid-depth of the beam on the compression side, a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed $.04\ f'_c$ on plain bars nor $.05\ f'_c$ on deformed bars.

4. Standard hook (see Section 906(a)), considered as developing 10,000 p.s.i., plus embedment sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified in Table 305(a). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

(*b*) The extreme ends of bars forming simple U or multiple stirrups shall be anchored by one of the methods of Section 904(a) or

shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least twelve diameters of the stirrup bar.

(*c*) The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees, or by being welded or otherwise rigidly attached thereto.

(*d*) Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

1. As specified in Section 904(a), (3).

2. As specified in Section 904(a), (4).

3. By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam, plus an extension of the bar of not less than twelve diameters of the bar terminating in a standard hook. This short radius bend extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

4. By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

5. The tensile unit stress at the beginning of a bend may be increased from 10,000 p.s.i. when the radius of bend is two bar diameters, at the rate of 1,000 p.s.i. tension for each increase of $1\frac{1}{2}$ bar diameters in the radius of bend, provided that the length of the bar in the bend and extension is sufficient to develop this increased tensile stress by bond at the unit stresses given in Section 904(e), (3).

(*f*) In all cases web reinforcement shall be carried as close to the compression surface of the beam as fireproofing regulations and the proximity of other steel will permit.

905—Anchorage of bars in footing slabs

(*a*) All bars in footing slabs shall be anchored by means of standard hooks. The outer faces of these hooks shall be not less than three inches nor more than six inches from the face of the footing.

906—Hooks

(*a*) The terms "hook" or "standard hook" as used herein shall mean either

1. A complete semicircular turn with a radius of bend on the axis of the bar of not less than three and not more than six bar diameters, plus an extension of at least four bar diameters at the free end of the bar, or

2. A 90° bend having a radius of not less than four bar diameters plus an extension of twelve bar diameters.

Hooks having a radius of bend of more than six bar diameters shall be considered merely as extensions to the bars, and shall be treated as in Section 904(e), (5).

(*b*) In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(*c*) No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 10,000 p.s.i.

(*d*) Hooks shall not be considered effective in adding to the compressive resistance of bars.

(*e*) Any mechanical device capable of developing the strength of the bar without damage to the concrete may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

CHAPTER 10—FLAT SLABS—WITH SQUARE OR RECTANGULAR PANELS

1000—Notation

A = The distance from the center line of the column, in the direction of any span, to the intersection of a 45-degree diagonal line from the center of the column to the bottom of the flat slab or drop panel, where such line lies wholly within the column, capital, or bracket, provided such capital or bracket is structurally capable of resisting shears and moments without excessive unit stress. In no case shall A be greater than one-eighth the span in the direction considered.

A_{av} = Average of the two values of A for the two columns at the ends of a column strip, in the direction of the spans considered.

c = Diameter or width of column capital at the under side of the slab or drop panel. No portion of the column capital shall be considered for structural purposes which lies outside the largest right circular cone, with 90 degrees vertex angle, that can be included within the outlines of the column capital.

L = Span length of slab center to center of columns in the direction of which bending is considered.

M_o = Sum of the positive and the average negative bending moments at the critical design sections of a flat slab panel. See Section 1003(b).

W = Total dead and live load uniformly distributed over a single panel area.

W_{av} = The average of the total load on two adjacent panels.

x = Coefficient of span L which gives the distance from the center of column to the critical section for negative bending in design according to Section 1002(a).

1001—Scope

(*a*) The term flat slab shall mean a reinforced concrete slab supported by columns with or without flaring heads or column capitals, with or without depressed or drop panels and generally without beams or girders.

(*b*) Recesses or pockets in flat slab ceilings, located between reinforcing bars and forming cellular or two-way ribbed ceilings, whether

left open or filled with permanent fillers, shall not prevent a slab from being considered a flat slab; but allowable unit stresses shall not be exceeded.

(*c*) This chapter provides for two methods of design of flat slab structures.

1. Any type of flat slab construction may be designed by application of the principles of continuity, using the method outlined in Section 1002, or using other recognized methods of elastic analysis. In either case, the design must be subject to the provisions of Sections 1005(a) and (c), 1006, 1008 and 1009.

2. The common cases of flat slab construction described in Section 1003 may be designed by the use of moment coefficients, given in Sections 1003 and 1004, and subject to the provisions of Sections 1005, 1006, 1007, 1008 and 1009.

1002—Design of flat slabs as continuous frames

(*a*) Except in the cases of flat slab construction where specified coefficients for bending may be used, as provided in Section 1003, bending and shear in flat slabs and their supports shall be determined by an analysis of the structure as a continuous frame, and all sections shall be proportioned to resist the moments and shears thus obtained. In the analysis, the following assumptions may be made:

1. The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported slabs, each strip bounded laterally by the center line of the panel on either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.

2. Each such bent may be analyzed in its entirety; or each floor thereof and the roof may be analyzed separately with its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom beyond which the slab continues.

3. The joints between columns and slabs may be considered rigid and this rigidity may be assumed to extend in the slabs a distance A from the center of the columns, and in the column to the intersection of the sides of the column and the 45 degree line defining A. The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and shear.

4. The supporting columns may be assumed free from settlement or lateral movement unless the amount thereof can be reasonably determined.

5. The moment of inertia of slab or column at any cross-section may be assumed to be that of the gross section of the concrete. Variation in the moments of inertia of the slabs and columns along their axes shall be taken into account.

6. Where the load to be supported is definitely known, the structure shall be analyzed for that load. Where the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum bending may be assumed to obtain at all sections under full live load. Elsewhere, maximum positive bending near mid-span of a panel may be assumed to obtain under full live load in the panel and in alternate panels; and maximum negative bending at a support may be assumed to obtain under full live load in the adjacent panels only.

7. Where neither beams nor girders help to transfer the slab load to the supporting column, the critical section for negative bending may be assumed as not more than the distance xL from the column center, where

$$x = 0.073 + 0.57 \frac{A}{L} \quad \ldots\ldots (17)$$

In slabs supported by beams, girders, or walls, the critical section for negative bending shall be assumed at the face of such support.

8. The numerical sum of the maximum positive and the average maximum negative bending moments for which provision is made in the design in the direction of either side of a rectangular panel shall be assumed as not less than

$$\frac{1}{10} W_{av} L \left(1 - \frac{4A_{av}}{3L}\right)^2 \quad \ldots\ldots (18)$$

9. The bending at critical sections across the slabs of each bent may be apportioned between the column strip and middle strip, as defined in Section 1005, in the ratio of the specified coefficients which affect such apportionment in the special cases of flat slabs provided for in Section 1003.

10. The maximum bending in columns may be assumed to obtain under full live load in alternate panels. Columns shall be proportioned to resist the maximum bending combined with the maximum direct load consistent therewith; and for maximum direct load combined with the bending under full load, the direct load subject to allowable reductions, in the manner provided in Chapter 11. In computing moments in columns at any floor, the far ends of the columns may be considered fixed.

(*b*) The foregoing provisions outline the method to be followed in analyzing and designing flat slabs in the general case. In all instances the design must conform to the requirements for panel strips and critical design sections, slab thickness and drop panels, capitals and brackets, arrangement of reinforcement and openings in flat slabs, as provided in Sections 1005(a) and (c), 1006, 1008 and 1009.

1003—Design of flat slabs by moment coefficients

(*a*) In those cases of flat slab construction which fall within the following limitations as to continuity and dimensions, the bending moments at critical sections may be determined by the use of specified coefficients as provided in Section 1004.

1. The ratio of length to width of panel does not exceed 1.33.
2. The slab is continuous for at least three panels in each direction.
3. The successive span lengths in each direction differ by not more than twenty per cent of the shorter span.

(*b*) In such slabs, the numerical sum of the positive and negative bending moments in the direction of either side of an interior rectangular panel shall be assumed as not less than

$$M_o = 0.09\,WL\left(1 - \frac{2c}{3L}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

(*c*) Three-fourths of the width of the strip shall be taken as the width of the section in computing compression due to bending, except that, on a section through a drop panel, three-fourths of the width of the drop panel shall be taken. Account shall be taken of any recesses which reduce the compressive area. Tension reinforcement distributed over the entire strip shall be included in the computations.

(*d*) The design of slabs under the procedure given in this section is subject to the provisions of all subsequent sections of this chapter (Sections 1004 to 1009).

1004—Bending moment coefficients

(*a*) The bending moments at the critical sections of the middle and column strips of an interior panel shall be assumed as given in Table 1004(a).

TABLE 1004(a)—BENDING MOMENTS IN INTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Negative moment	$0.50M_o$
	Positive moment	$0.20M_o$
Middle strip	Negative moment	$0.15M_o$
	Positive momtnt	$0.15M_o$
Without drop panel		
Column strip	Negative moment	$0.46M_o$
	Positive moment	$0.22M_o$
Middle strip	Negative moment	$0.16M_o$
	Positive moment	$0.16M_o$

TABLE 1004(b)—BENDING MOMENTS IN EXTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Exterior negative	$0.45M_o$
	Positive moment	$0.25M_o$
	Interior negative	$0.55M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.19M_o$
	Interior negative	$0.165M_o$
Without drop panel		
Column strip	Exterior negative	$0.41M_o$
	Positive moment	$0.28M_o$
	Interior negative	$0.50M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.20M_o$
	Interior negative	$0.176M_o$

TABLE 1004(c)—BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS

		Marginal Beams with Depth greater than 1½ times the Slab Thickness; or Bearing Wall.		Marginal Beams with depth 1½ times the Slab Thickness or less.	
(a) Load to be carried by Marginal Beam or Wall		Loads directly superimposed upon it plus a uniform load equal to one-quarter of the total live and dead panel load.		Loads directly superimposed upon it exclusive of any panel load.	
		With Drop	Without Drop	With Drop	Without Drop
(b) Moment to be used in the design of Half Column Strip adjacent and parallel to Marginal Beam or Wall.	Neg.	$0.125M_o$	$0.115M_o$	$0.25M_o$	$0.23M_o$
	Pos.	$0.05M_o$	$0.055M_o$	$0.10M_o$	$0.11M_o$
(c) Negative Moment to be used in Design of Middle Strip continuous across a Beam or Wall	Neg.	$0.195M_o$	$0.208M_o$	$0.15M_o$	$0.16M_o$

(*b*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the

exterior supports consist of reinforced concrete columns or reinforced concrete bearing walls integral with the slab, the ratio of stiffness of the support to that of the slab being at least as great as the ratio of the live load to the dead load and not less than one, shall be assumed as given in Table 1004(b). Where a flat slab is so supported by a wall providing restraint at the discontinuous edge, the coefficient for negative bending at the edge shall be assumed more nearly equal in the column and middle strips, the sum remaining as given in Table 1004(b), but that for the column strip shall not be less than 0.30 M_o. Bending in middle strips parallel to a discontinuous edge, except in a corner panel, shall be assumed the same as in an interior panel. M_o shall be determined as provided in Section 1003(b) for an interior panel.

(*c*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports are masonry bearing walls or other construction which provide only negligible restraint to the slab, shall be assumed as given in Table 1004(b) with the following modifications.

1. On critical sections at the face of the exterior support, negative bending in each strip shall be assumed as 0.05 M_o.

2. The coefficients for positive bending shall be increased by forty per cent.

3. The coefficients for negative bending at the first interior columns shall be increased thirty per cent.

(*d*) The bending moments in panels with marginal beams or walls, in the strips parallel and close thereto, and in the beams, shall be determined upon the basis of assumptions presented in Table 1004(c).

(*e*) For design purposes any of the moment coefficients of Tables 1004(a), 1004(b), and 1004(c) may be varied by not more than six per cent, but the numerical sum of the positive and negative moments in a panel shall not be taken as less than the amount specified.

(*f*) Panels supported by marginal beams on opposite edges shall be designed as solid one or two-way slabs to carry the entire panel load.

(*g*) The ratio of reinforcement in any strip shall not be less than 0.0025.

1005—Panel strips and critical design sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one half panel in width, symmetrical about panel center line and extending through the panel in the direction of the span for bending.

A column strip consisting of the two adjacent quarter-panels either side of the column center lines.

(*b*) The critical sections for bending are located as follows:

Sections for negative bending shall be taken along the edges of the panel, on column center lines between capitals and around the perimeters of column capitals.

Sections for positive bending shall be taken at mid-span of the strips.

(*c*) Only the reinforcement which crosses a critical section within a strip may be considered effective to resist bending in the strip at that section. Reinforcement which crosses such section at an angle with the center-line of the strip shall be assumed to contribute to the resistance of bending only its effective area in the direction of the strip, as defined in Chapter 1.

1006—Slab thickness and drop panels

(*a*) The thickness of a flat slab and the size and thickness of the drop panel, where used, shall be such that the compressive stress due to bending at the critical sections of any strip and the shear about the column capital and the drop panel shall not exceed the unit stresses allowed in concrete of the quality used.

(*b*) The shearing stresses in the slab outside the capital or drop panel shall be computed as provided in Section 807.

(*c*) Slab thickness shall not, however, be less than

$$\frac{L}{40} \text{ with drop panels}$$

or

$$\frac{L}{36} \text{ without drop panels}$$

(*d*) The thickness of the drop panel below the slab shall not be more than one-fourth the distance from the edge of the column capital to the edge of the drop panel.

1007—Capitals and brackets

(*a*) Where a column is without a flaring concrete capital the distance *c* shall be taken as the diameter of the column. Structural metal embedded in the slab or drop panel may be regarded as contributing to resistance in bending and shear.

(*b*) Where a reinforced concrete beam frames into a column without capital or bracket on the same side with the beam, the value of c may be taken as the width of the column plus twice the projection of the beam above or below the slab or drop panel for computing bending in strips parallel to the beam.

(*c*) Brackets capable of transmitting the negative bending and the shear in the column strips to the columns without excessive unit stress may be substituted for column capitals at exterior columns. The value of c where brackets are used shall be taken as twice the distance from the center of the column to a point where the bracket is $1\frac{1}{2}$ inches thick, but not more than the thickness of the column plus twice the depth of the bracket.

(*d*) The average of the diameters c of the column capitals at the four corners of a panel shall be used in determining the bending in the middle strips of the panel. The average of the diameters c of the two column capitals at the ends of a column strip shall be used in determining bending in the strip.

1008—Arrangement of reinforcement

(*a*) Slab reinforcement shall be provided to resist the bending and bond stresses not only at critical sections, but also at intermediate sections.

(*b*) Bars shall be spaced evenly across strips or bands and the spacing shall not exceed three times the slab thickness.

(*c*) In exterior panels the reinforcement perpendicular to the discontinuous edge for positive bending, shall extend to the edge and have embedment of at least six inches in spandrel beams, walls or columns. All such reinforcement for negative bending shall be bent, hooked or otherwise anchored in spandrel beams, walls or columns.

1009—Openings in flat slabs

Openings of any size may be cut through a flat slab if provision is made for the total positive and negative resisting moments, as required in Sections 1002 or 1003, without exceeding the allowable stresses as given in Sections 305 and 306.

CHAPTER 11—REINFORCED CONCRETE COLUMNS AND WALLS

1100—Notation

A_c = Area of core of a spirally reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

C = Ratio of allowable concrete stress, f_a, in axially loaded column to allowable fiber stress for concrete in flexure.

$D = \frac{l^2}{2R^2}$ = a factor, usually varying from 3 to 9. (The term R as used here is the radius of gyration of the entire column section.)

d = The least lateral dimension of a concrete column.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$F = \frac{\textit{Yield point of pipe}}{45{,}000}$ (See Section 1106(b)).

f_a = Average allowable stress in the concrete of an axially loaded reinforced concrete column.

f_c = Computed concrete fiber stress in an eccentrically loaded column.

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

f_p = Maximum allowable concrete fiber stress in an eccentrically loaded column.

f_r = Allowable unit stress in the metal core of a composite column.

f'_r = Allowable unit stress on unencased steel columns and pipe columns.

f_s = Nominal working stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

h = Unsupported length of column.

K = Least radius of gyration of a metal pipe section (in pipe columns).

$n = \dfrac{30{,}000}{f'_c}$

N = Axial load applied to reinforced concrete column.

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g.

P = Total allowable axial load on a column whose length does not exceed ten times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

R = Least radius of gyration of a section.

t = Overall depth of column section.

1101—Limiting dimensions

(*a*) The following sections on reinforced concrete and composite columns, except Section 1107(a), apply to a short column for which the unsupported length is not greater than ten times the least dimension. When the unsupported length exceeds this value, the design shall be modified as shown in Section 1107 (a). Principal columns in buildings shall have a minimum diameter of twelve inches, or in the case of rectangular columns, a minimum thickness of ten inches, and a minimum gross area of 120 sq. in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of six inches.

1102—Unsupported length of columns

(*a*) For purposes of determining the limiting dimensions of columns, the unsupported length of reinforced concrete columns shall be taken as the clear distance between floor slabs, except that

1. In flat slab construction, it shall be the clear distance between the floor and the lower extremity of the capital.

2. In beam and slab construction, it shall be the clear distance between the floor and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

3. In columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane; provided that to be an adequate support, two such struts shall meet the column at approximately the same level, and the angle

between vertical planes through the struts shall not vary more than 15 degrees from a right angle. Such struts shall be of adequate dimensions and anchorage to restrain the column against lateral deflection.

4. In columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the column.

(*b*) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

1103—Spirally reinforced columns

(*a*) *Allowable Load*—The maximum allowable axial load, P, on columns with closely spaced spirals enclosing a circular concrete core reinforced with longitudinal bars shall be that given by Formula (20).

$$P = A_g\,(0.225\,f'_c + f_s p_g) \qquad (20)$$

Wherein A_g = the gross area of the column

f'_c = compressive strength of the concrete

f_s = nominal working stress in vertical column reinforcement, to be taken at forty per cent of the minimum specification value of the yield point; viz., 16,000 p.s.i. for intermediate grade steel and 20,000 p.s.i. for rail or hard grade steel.*

p_g = ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.

(*b*) *Vertical Reinforcement*—The ratio p_g shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be six, and the minimum diameter shall be $\frac{5}{8}$ in. The center to center spacing of bars within the periphery of the column core shall not be less than $2\frac{1}{2}$ times the diameter for round bars or three times the side dimension for square bars. The clear spacing between bars shall not be less than $1\frac{1}{2}$ inches or $1\frac{1}{2}$ times the maximum size of the coarse aggregate used. These spacing rules also apply to adjacent pairs of bars at a lapped splice; each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars.

(*c*) *Splices in Vertical Reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

*Nominal working stresses for reinforcement of higher yield point may be established at forty per cent of the yield point stress, but not more than 30,000 p.s.i., when the properties of such reinforcing steels have been definitely specified by standards of A.S.T.M. designation. If this is done, the lengths of splice required by Section 1103(c) shall be increased accordingly.

1. For deformed bars—with concrete having a strength of 3000 p.s.i. or above, twenty-four diameters of bar of intermediate grade steel and thirty diameters of bar of hard grade steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the nominal working stress. When the concrete strengths are less than 3000 p.s.i., the amount of lap shall be one-third greater than the values given above.

2. For plain bars—the minimum amount of lap shall be twenty-five per cent greater than that specified for deformed bars.

3. Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds 1¼ in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

4. Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded by a concrete capital, floor slab or by metal ties or reinforcing spirals. Where bars are offset, the slope of the inclined portion from the axis of the column shall not exceed 1 in 6 and the bars above and below the offset shall be parallel to the axis of the column.

(*d*) *Spiral Reinforcement*—The ratio of spiral reinforcement, p', shall not be less than the value given by Formula (21).

$$p' = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f'_s} \qquad (21)$$

Wherein p' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

f'_s = useful limit stress of spiral reinforcement, to be taken as 40,000 p.s.i. for hot rolled rods of intermediate grade, 50,000 p.s.i. for rods of hard grade, and 60,000 p.s.i. for cold drawn wire.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spirals shall be of such size and so assembled as to permit handling and placing without being distorted from the designed dimensions. The material used in spirals shall have a minimum diameter of ¼ in. for rolled bars or No. 4 W. & M. gage for drawn wire. Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral rod or wire at each end of the spiral unit. Splices, when

necessary shall be made in spiral rod or wire by welding or by a lap of 1½ turns. The center to center spacing of the spirals shall not exceed one-sixth of the core diameter. The clear spacing between spirals shall not exceed 3 in. nor be less than 1⅜ in. or 1½ times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the floor level in any story or from the top of the footing in the basement, to the level of the lowest horizontal reinforcement in the slab, drop panel or beam above. In a column with a capital, it shall extend to a plane at which the diameter or width of the capital is twice that of the column.

(*e*) *Protection of Reinforcement*—The column reinforcement shall be protected everywhere by a covering of concrete cast monolithically with the core, for which the thickness shall not be less than 1½ in. nor less than 1½ times the maximum size of the coarse aggregate, nor shall it be less than required by the fire protection and weathering provisions of Section 507.

(*f*) *Isolated Column with Multiple Spirals*—In case two or more interlocking spirals are used in a column, the outer boundary of the column shall be taken as a rectangle of which the sides are outside the extreme limits of the spiral at a distance equal to the requirements of Section 1103(*e*).

(*g*) *Limits of Section of Column Built Monolithically with Wall*—For a spiral column built monolithically with a concrete wall or pier, the outer boundary of the column section shall be taken either as a circle at least 1½ in. outside the column spiral or as a square or rectangle of which the sides are at least 1½ in. outside the spiral or spirals.

(*h*) *Equivalent Circular Columns*—As an exception to the general procedure of utilizing the full gross area of the column section, it shall be permissible to design a circular column and to build it with a square, octagonal, or other shaped section of the same least lateral dimension. In such case, the allowable load, the gross area considered, and the required percentages of reinforcement shall be taken as those of the circular column.

1104—Tied columns

(*a*) *Allowable Load*—The maximum allowable axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80 per cent of that given by Formula (20). The ratio, p_g, to be considered in tied columns shall not be less than 0.01 nor more than 0.04. The longitudinal reinforcement shall consist of at least four bars, of minimum diameter of ⅝ inch. Splices in reinforcing bars shall be made as described in Section 1103 (c).

(*b*) *Lateral Ties*—Lateral ties shall be at least ¼ in. in diameter and shall be spaced apart not over 16 bar diameters, 48 tie diameters or the least dimension of the column. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar is held firmly in its designed position and has lateral support equivalent to that provided by a 90-degree corner of a tie.

(*c*) *Limits of Column Section*—In a tied column which for architectural reasons has a larger cross section than required by considerations of loading, a reduced effective area, A_g, not less than one-half of the total area may be used in applying the provisions of Section 1104 (a).

1105—Composite columns

(*a*) *Allowable Load*—The allowable load on a composite column, consisting of a structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by Formula (22).

$$P = 0.225\,A_c f'_c + f_s A_s + f_r A_r \quad \ldots\ldots (22)$$

Wherein A_c = net area of concrete section
$= A_g - A_s - A_r$
A_s = cross-sectional area of longitudinal bar reinforcement.
A_r = cross-sectional area of the steel or cast-iron core.
f_r = allowable unit stress in metal core, not to exceed 16,000 p.s.i. for a steel core; or 10,000 p.s.i. for a cast-iron core.

The remaining notation is that of Section 1103.

(*b*) *Details of Metal Core and Reinforcement*—The cross-sectional area of the metal core shall not exceed 20 per cent of the gross area of the column. If a hollow metal core is used it shall be filled with concrete. The amounts of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shell outside the spiral shall conform to the limiting values specified in Section 1103 (b), (c) and (d). A clearance of at least three inches shall be maintained between the spiral and the metal core at all points except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to two inches.

(*c*) *Splices and Connections of Metal Cores*—Metal cores in composite columns shall be accurately milled at splices and positive provision shall be made for alignment of one core above another. At the column base, provision shall be made to transfer the load to the footing at safe unit stresses in accordance with Section 305 (a). The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal

as to leave ample section of concrete above the base for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members such as billets, brackets or other positive connections; these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of Formula (22) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, in accordance with Formula (20), all floor loads brought onto the column at levels between the metal brackets or connections. In applying Formula (20), the value of A_g shall be interpreted as the area of the concrete section outside the metal core, and the allowable load on the reinforced concrete section shall be further limited to $0.35\ f'_c A_g$. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders.

(*d*) *Allowable Load on Metal Core Only*—The metal cores of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

1106—Combination columns

(*a*) *Steel Columns Encased in Concrete*—The allowable load on a structural steel column which is encased in concrete at least 2½ inches thick over all metal (except rivet heads) reinforced as hereinafter specified, shall be computed by Formula (23).

$$P = A_r f'_r \left[1 + \frac{A_g}{100\ A_r}\right] \qquad (23)$$

Wherein A_r = cross-sectional area of steel column.
f'_r = allowable stress for unencased steel column.
A_g = total area of concrete section.

The concrete used shall develop a compressive strength, f'_c, of at least 2000 p.s.i. at 28 days. The concrete shall be reinforced by the equivalent of welded wire mesh having wires of No. 10 W. and M. gage, the wires encircling the column being spaced not more than four inches apart and those parallel to the column axis not more than eight inches apart. This mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface and shall be lap-spliced at least forty wire diameters and wired at the splice. Special brackets shall be used to receive the entire floor load at each floor level. The steel column shall be designed to carry safely any construction or other loads to be placed upon it prior to its encasement in concrete.

(*b*) *Pipe Columns*—The allowable load on columns consisting of steel pipe filled with concrete shall be determined by Formula (24).

$$P = 0.225 f'_c A_c + f'_r A_r \quad \ldots\ldots (24)$$

The value of f'_r shall be given by Formula (25).

$$f'_r = \left[18{,}000 - 70 \frac{h}{K}\right] F \quad \ldots\ldots (25)$$

Wherein f'_r = allowable unit stress in metal pipe.
h = unsupported length of column.
K = least radius of gyration of metal pipe section.

$$F = \frac{\textit{yield point of pipe.}}{45{,}000}$$

If the yield point of the pipe is not known, the factor F shall be taken as 0.5.

1107—Long columns

(*a*) The maximum allowable load, P', on axially loaded reinforced concrete or composite columns having a length, h, greater than ten times the least lateral dimension, d, shall be given by Formula (26).

$$P' = P\left[1.3 - .03 \frac{h}{d}\right] \quad \ldots\ldots (26)$$

where P is the allowable axial load on a short column as given by Formulas (20) and (22).

The maximum allowable load, P', on eccentrically loaded columns in which $\frac{h}{d}$ exceeds ten shall also be given by Formula (26), in which P is the allowable eccentrically applied load on a short column as determined by the provisions of Sections 1109 and 1110. In long columns subjected to definite bending stresses, as determined in Section 1108, the ratio $\frac{h}{d}$ shall not exceed twenty.

1108—Bending moments in columns

(*a*) The bending moments in the columns of all reinforced concrete structures shall be determined on the basis of loading conditions and restraint and shall be provided for in the design. When the stiffness and strength of the columns are utilized to reduce moments in beams, girders, or slabs, as in the case of rigid frames, or in other forms of continuous construction wherein column moments are unavoidable, they shall be provided for in the design. In building frames, particular

attention shall be given to the effect of unbalanced floor loads on both exterior and interior columns and of eccentric loading due to other causes. Wall columns shall be designed to resist moments produced by

1. Loads on all floors of the building
2. Loads on a single exterior bay at two adjacent floor levels, or
3. Loads on a single exterior bay at one floor level

Resistance to bending moments at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffnesses and conditions of restraint.

1109—Determination of combined axial and bending stresses

(*a*) In a reinforced concrete column, designed by the methods of this Chapter, which is (1) symmetrical about two perpendicular planes through its axis and (2) subject to an axial load, N, combined with bending in one or both of the planes of symmetry (but with the ratio of eccentricity to depth, e/t, no greater than 1.0 in either plane), the combined fiber stress in compression may be computed on the basis of recognized theory applying to uncracked sections, using Formula 27.

$$f_c = \frac{N}{A_g}\left[\frac{1 + \frac{De}{t}}{1 + (n-1)p_g}\right] \quad \text{.....(27)}$$

Equating this calculated stress, f_c, to the allowable stress, f_p, in Formula 29, it follows that the column can be designed for an equivalent axial load, P, as given by Formula 28.*

$$P = N\left[1 + \frac{CDe}{t}\right] \quad \text{.....(28)}$$

When bending exists on both axes of symmetry, the quantity $\frac{De}{t}$ is to be computed as the numerical sum of the $\frac{De}{t}$ quantities in the two directions.

(*b*) For columns in which the load, N, has an eccentricity, e, greater than the column depth, t, or for beams subject to small axial loads, the determination of the fiber stress f_c shall be made by use of recognized theory for cracked sections, based on the assumption that no tension exists in the concrete. For such cases the tensile steel stress shall also be investigated.

*For approximate or trial computations, D may be taken as eight for a circular spiral column and as five for a rectangular tied or spiral column.

1110—Allowable combined axial and bending stress

(*a*) For spiral and tied columns, eccentrically loaded or otherwise subjected to combined axial compression and flexural stress, the maximum allowable compressive stress, f_p, is given by Formula (29).

$$f_p = f_a \left[\frac{1 + \frac{De}{t}}{1 + \frac{CDe}{t}} \right] = f_a \left[\frac{t + De}{t + CDe} \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Wherein the notation is that of Section 1103 and 1109, and, in addition f_a is the average allowable stress in the concrete of an axially loaded reinforced concrete column, and C is the ratio of f_a to the allowable fiber stress for members in flexure. Thus $f_a = \frac{0.225\, f'_c + f_s p_g}{1 + (n - 1)\, p}$ for spiral columns and 0.8 of this value for tied columns, and $C = \frac{f_a}{0.45\, f'_c}$.

1111—Wind stresses

(*a*) When the allowable stress in columns is modified to provide for combined axial load and bending, and the stress due to wind loads is also added, the total shall still come within the allowable values specified for wind loads in Section 603 (c).

1112—Reinforced concrete walls

(*a*) The allowable working stresses in reinforced concrete bearing walls with minimum reinforcement as required by Section 1112(i), shall be $0.25f'_c$ for walls having a ratio of height to thickness of ten or less, and shall be reduced proportionally to $0.15f'_c$ for walls having a ratio of height to thickness of twenty-five. When the reinforcement in bearing walls is designed, placed and anchored in position as for tied columns, the allowable working stresses shall be on the basis of Section 1104, as for columns. In the case of concentrated loads, the length of the wall to be considered as effective for each shall not exceed the center to center distance between loads, nor shall it exceed the width of the bearing plus four times the wall thickness. The ratio p_g shall not exceed 0.04.

(*b*) Walls shall be designed for any lateral or other pressure to which they are subjected. Proper provision shall be made for eccentric loads and wind stresses. In such designs the allowable stresses shall be as given in Section 305(a) and 603(c).

(*c*) Panel and enclosure walls of reinforced concrete shall have a thickness of not less than five inches and not less than one thirtieth the distance between the supporting or enclosing members.

(*d*) Bearing walls of reinforced concrete in building of fire-resistive construction shall be not less than six inches in thickness for the uppermost fifteen feet of their height; and for each successive twenty-five feet downward, or fraction thereof, the minimum thickness shall be increased one inch. In two story dwellings the walls may be six inches in thickness throughout.

(*e*) In buildings of non-fire resistive construction bearing walls of reinforced concrete shall not be less than one and one-third times the thickness required for buildings of fire-resistive construction, except that for dwellings of two stories or less in height the thickness of walls may be the same as specified for buildings of fire-resistive construction.

(*f*) Exterior basement walls, foundation walls, fire walls and party walls shall not be less than eight inches thick whether reinforced or not.

(*g*) Reinforced concrete bearing walls shall have a thickness of at least one twenty-fifth of the unsupported height or width, whichever is the shorter; provided however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of increased thickness.

(*h*) Reinforced concrete walls shall be anchored to the floors, columns, pilasters, buttresses and intersecting walls with reinforcement at least equivalent to three-eighths inch round bars twelve inches on centers, for each layer of wall reinforcement.

(*i*) Reinforced concrete walls shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall, if of bars, and 0.0018 times the area if of electrically welded wire fabric.* The wire of the welded fabric shall be of not less than No. 10 W. & M. gage. Walls more than ten inches in thickness shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than two inches nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement, shall be placed not less than three-fourths inches and not more than one-third the thickness of the wall from the interior surface. Bars, if used, shall not be less than the equivalent of three-eighths inch round

*Expanded metal has been omitted until a specification can be formulated.

bars, nor shall they be spaced more than eighteen inches on centers. Welded wire* reinforcement for walls shall be in flat sheet form.

(*j*) In addition to the minimum as prescribed in 1112(i) there shall be not less than two five-eighths inch diameter bars around all window or door openings. Such bars shall extend at least twenty-four inches beyond the corner of the openings.

(*k*) Where reinforced concrete bearing walls consist of studs or ribs tied together by reinforced concrete members at each floor level, the studs may be considered as columns, but the restrictions as to minimum diameter or thickness of columns shall not apply.

*See footnote previous page.

CHAPTER 12—FOOTINGS

1201—Scope

(*a*) The requirements prescribed in Sections 1202 to 1209 apply only to isolated footings.*

1202—Loads and reactions

(*a*) Footings shall be proportioned to sustain the applied loads and induced reactions without exceeding the allowable stresses as prescribed in Sections 305 and 306, and as further provided in Sections 1205, 1206 and 1207.

(*b*) In cases where the footing is concentrically loaded and the member being supported does not transmit any moment to the footing, computations for moments and shears shall be based on an upward reaction assumed to be uniformly distributed per unit area or per pile and a downward applied load assumed to be uniformly distributed over the area of the footing covered by the column, pedestal, wall, or metallic column base.

(*c*) In cases where the footing is eccentrically loaded and/or the member being supported transmits a moment to the footing, proper allowance shall be made for any variation that may exist in the intensities of reaction and applied load consistent with the magnitude of the applied load and the amount of its actual or virtual eccentricity.

(*d*) In the case of footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the center of the pile.

1203—Sloped or stepped footings

(*a*) In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that the allowable stresses are not exceeded at any section.

(*b*) In sloped or stepped footings, the effective cross-section in compression shall be limited by the area above the neutral plane.

(*c*) Sloped or stepped footings shall be cast as a unit.

1204—Bending moment

(*a*) The external moment on any section shall be determined by passing through the section a vertical plane which extends completely

*The committee is not prepared at this time to make recommendations for combined footings—those supporting more than one column or wall.

across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of said plane.

(*b*) The greatest bending moment to be used in the design of an isolated footing shall be the moment computed in the manner prescribed in Section 1204 (a) at sections located as follows:

1. At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.

2. Halfway between the middle and the edge of the wall, for footings under masonry walls.

3. Halfway between the face of the column or pedestal and the edge of the metallic base, for footings under metallic bases.

(*c*) The width resisting compression at any section shall be assumed as the entire width of the top of the footing at the section under consideration.

(*d*) In one-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to the moment computed in the manner prescribed in Section 1204(a); and the reinforcement thus determined shall be distributed uniformly across the full width of the section.

(*e*) In two-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to eighty-five per cent of the moment computed in the manner prescribed in Section 1204(a); and the total reinforcement thus determined shall be distributed across the corresponding resisting section in the manner prescribed for square footings in Section 1204(f), and for rectangular footings in Sec. 1204(g).

(*f*) In two-way square footings, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

(*g*) In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. In the case of the reinforcement in the short direction, that portion determined by formula (30) shall be uniformly distributed across a band-width (*B*) centered with respect to the center line of the column or pedestal and having a width equal to the length of the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

$$\frac{\textit{Reinforcement in band-width } (B)}{\textit{Total reinforcement in short direction}} = \frac{2}{(S+1)} \qquad (30)$$

In formula (30), "*S*" is the ratio of the long side to the short side of the footing.

1205—Shear and bond

(*a*) The critical section for shear to be used as a measure of diagonal tension shall be assumed as a vertical section obtained by passing a series of vertical planes through the footing, each of which is parallel to a corresponding face of the column, pedestal, or wall and located a distance therefrom equal to the depth d for footings on soil, and one-half the depth d for footings on piles.

(*b*) Each face of the critical section as defined in Section 1205(a) shall be considered as resisting an external shear equal to the load on an area bounded by said face of the critical section for shear, two diagonal lines drawn from the column or pedestal corners and making 45° angles with the principal axes of the footing, and that portion of the corresponding edge or edges of the footing intercepted between the two diagonals.

(*c*) Critical sections for bond shall be assumed at the same planes as those prescribed for bending moment in Section 1204(b); also at all other vertical planes where changes of section or of reinforcement occur.

(*d*) Computations for shear to be used as a measure of bond shall be based on the same section and loading as prescribed for bending moment in Section 1204(a).

(*e*) The total tensile reinforcement at any section shall provide a bond resistance at least equal to the bond requirement as computed from the following percentages of the external shear at the section:

1. In one-way reinforced footings, 100 per cent.
2. In two-way reinforced footings, 85 per cent.

(*f*) In computing the external shear on any section through a footing supported on piles, the entire reaction from any pile whose center is located six inches or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose center is located six inches or more inside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight-line interpolation between full value at six inches outside the section and zero value at six inches inside the section.

(*g*) For allowable shearing stresses, see Section 305 and 808.

(*h*) For allowable bond stresses, see Section 305 and 901 to 905.

1206—Transfer of stress at base of column

(*a*) The stress in the longitudinal reinforcement of a column or pedestal shall be transferred to its supporting pedestal or footing

either by extending the longitudinal bars into the supporting member, or by dowels.

(*b*) In case the transfer of stress in the reinforcement is accomplished by extension of the longitudinal bars, they shall extend into the supporting member the distance required to transfer to the concrete, by allowable bond stress, their full working value.

(*c*) In cases where dowels are used, their total sectional area shall be not less than the sectional area of the longitudinal reinforcement in the member from which the stress is being transferred. In no case shall the number of dowels per member be less than four and the diameter of the dowels shall not exceed the diameter of the column bars by more than one-eighth inch.

(*d*) Dowels shall extend up into the column or pedestal a distance at least equal to that required for lap of longitudinal column bars (see Section 1103) and down into the supporting pedestal or footing the distance required to transfer to the concrete, by allowable bond stress, the full working value of the dowel.

(*e*) The compressive stress in the concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The unit compressive stress on the loaded area shall not exceed the bearing stress allowable for the quality of concrete in the supporting member as limited by the ratio of the loaded area to the supporting area.

(*f*) For allowable bearing stresses see Table 305(a), Section 305.

(*g*) In sloped or stepped footings, the supporting area for bearing may be taken as the top horizontal surface of the footing, or assumed as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the area actually loaded, and having side slopes of one vertical to two horizontal.

1207—Pedestals and footings (plain concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on sections as prescribed in Section 1204 and 1205 for reinforced concrete footings.

1208—Footings supporting round columns

(*a*) In computing the stresses in footings which support a round or octagonal concrete column or pedestal, the "face" of the column or pedestal shall be taken as the side of a square having an area equal to the area enclosed within the perimeter of the column or pedestal.

1209—Minimum edge-thickness

(*a*) In reinforced concrete footings, the thickness above the reinforcement at the edge shall be not less than six in. for footings on soil, nor less than twelve in. for footings on piles.

(*b*) In plain concrete footings, the thickness at the edge shall be not less than eight in. for footings on soil, nor less than fourteen in. above the tops of the piles for footings on piles.

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ACI Standard

Recommended Practice for use of Metal Supports for Reinforcement—(ACI 319-42)*

Reported by ACI Committee 319

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1. General

All reinforcing steel shall be accurately located in the forms and held firmly in place before and during the placing of concrete, by means of metallic supports, spacer bars or wires.

2. Supports

Bar supports shall be sufficiently strong and stable to sustain construction loads without permanent distortion or displacement. (see "Types of Devices") These shall be designed to permit full embedment in the concrete without the formation of voids and with a minimum exposure of metal on finished surfaces. They shall, in combination with tie wires or clips, provide positive means for establishing and maintaining operative association with the reinforcing steel.

3. Number and location of supports

Minimum requirements for number and location of supports and spacers under various usual conditions shall be as set forth in Tables 1 to 4.

4. Responsibility

The number, type, and location of supporting and spacing devices shall be clearly indicated on the drawings which show placing of reinforcement.

It shall be the responsibility of the seller of such devices to furnish the quantities, types, and capacities as indicated on the drawings, with the necessary identification marks.

It shall be the responsibility of the builder to install all of such devices, and to maintain them accurately in place until the concrete is placed.

*Adopted as a Standard of the American Concrete Institute at its 38th Annual Convention; ratified by letter ballot May 24, 1942.

Any and all disturbances of reinforcement from any cause whatsoever shall be fully corrected prior to the placing of concrete, and all damaged bar supports and spacers shall be repaired, or removed and replaced by the builder.

TABLE 1.—ONE-WAY SLAB CONSTRUCTION

Span	Rows of Slabs Bar Spacers for Panel	High Chairs and ⅝″ Support Bars in Slabs 4″ and Thicker*	
		Support Bars	High Chairs
0′ to 6′ Steel Continuous	1	1 Row at Beam	4′-0 o.c.
0′ to 14′ Steel Not Continuous	2	2 Rows at Beam	4′-0 o.c.
14′ to 20′ Steel Not Continuous	3	2 Rows at Beam	4′-0 o.c.
20′ to 26′ Steel Not Continuous	4	2 Rows at Beam	4′-0 o.c.

*Continuous high chairs may be substituted for individual high chairs and support bars.

Table 2.—ORDINARY BEAM AND JOIST CONSTRUCTION

(Bars 1 in. square and smaller)

Clear spans, beam (or joist)	Number of beam (joist) chairs					
	Single layer of bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 14 ft.	2	2	2	2	2	2
Over 14 ft. to 23 ft.	3	3	2	3	2	2
Over 23 ft. to 30 ft.	4	4	3	4	3	2
Over 30 ft.	(*)	(*)	(*)	(*)	(*)	(*)

*See Table 3.

TABLE 3.—HEAVY BEAM AND GIRDER CONSTRUCTION

(Beams or girders with large number of 1⅛- or 1¼-inch bars)

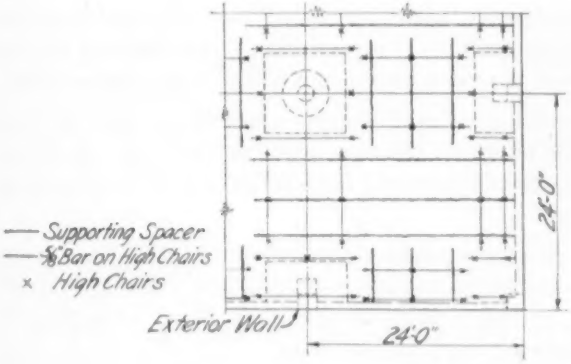
Clear Spans	Number of beam chairs					
	Single layer of Bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 14 ft.	2	3	2	3	2	2
Over 14 ft. to 23 ft.	3	5	2	6	2	2
Over 23 ft. to 30 ft.	4	6	3	7	3	2
Over 30 ft. to 40 ft.	6	7	4	8	4	3
Over 40 ft. to 50 ft.	7	8	4	9	5	4
Spacing of beam chairs in spans other than above: All spans	7′0″	6′0″	10′0″	5′0″	8′0″	10′0″

TABLE 4.—FLAT SLABS

(Two and four way flat slabs—See Fig. 1 and 2)

Spans (center to center of Columns)	Supporting Spacers		High Chairs and 5/8" Support Bars under Ends of Bent Bars*			
	Column Strip or Direct Band	Middle Strip or Diagonal Band Bottom Layer	Support for Column Head Reinforcement		Support for Negative Reinforcement, Middle Strip	
			Support Bars	High Chairs	Support Bars	High Chairs
Over 0 ft. to 18 ft.	3	2	....	....	....	....
Over 18 ft. to 26 ft.	3	3	....	....	....	....
Over 26 ft. to 36 ft.	4	4	....	....	....	....
Around interior columns	....	....	2	8	....	....
Around exterior columns	....	....	2	5	....	....
Around corner columns	....	....	2	4	....	....
In interior panels	....	....	....	....	4	12
In exterior panels	....	....	....	....	5	15
In corner panels	....	....	....	....	6	18

*Continuous high chairs may be substituted for individual high chairs and support bars.



In roof slabs use one (1) more supporting spacer under column strips, or diagonal bands, and bottom layers of middle strips or diagonal bands, and one (1) more 5/8in. support bar at interior column heads, than the number shown in Table 3.

Fig. 1—Two-way system

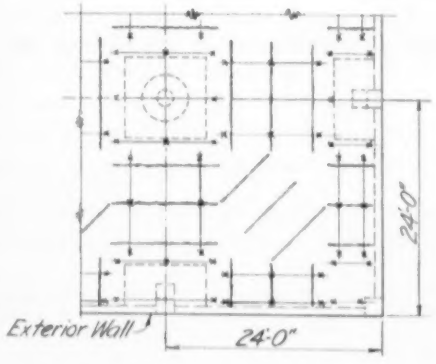


Fig. 2—Four-way system

TYPES OF DEVICES

Since various satisfactory types of supporting devices are on the market, this report makes no attempt to specify exact details and sizes. However, the following indicates the Committee's opinion as to a specification covering the most common types, which should produce devices meeting the requirements of paragraph 2, "Supports."

For supporting devices for slab steel in which the spanning member is a round section, either of cold drawn wire or of hot rolled steel having a yield point not less than 45,000 psi., the spacing of legs shall not exceed the following:

For continuous high chairs $s = 400\ d^3$

For continuous slab spacers, where the spanning member is not over $1\frac{1}{2}$ in. above the forms $s = 550\ d^3$

in which

s — spacing of legs in inches

d — diameter of round wire or rod in inches

The above rules shall apply to spacers having legs at a spacing not to exceed 30 in.

Supporting legs for continuous high chairs shall be so formed that the effective width of the support where it rests on the form shall not be less than $\frac{3}{4}$ of the height from the form to the top of the spanning member.

Supporting legs for continuous slab spacers shall be so formed that the effective width of the support, where it rests on the form, shall not be less than two times the height from the form to the top of the spanning member.

Legs of continuous high chairs made of wire or round rods shall be of such size that the ratio of height to radius of gyration shall not exceed 120 in the case of wires having a diameter less than $\frac{1}{4}$ in. or in case of all chairs which are supported upon metal forms.

In the case of continuous high chairs which are supported upon wood forms and in which the material of the legs is of a diameter of $\frac{1}{4}$ in. or larger the ratio of height to radius of gyration shall not exceed 156.

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ACI Standard

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)*

Reported by ACI Committee 614

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I. INTRODUCTION

1. This report has been prepared in response to an expressed need for an outline of good practices for measuring and mixing the ingredients for concrete and for placing the finished product.

2. At the outset the Committee faced the question whether it should describe "common practices" or "best practices." Since this report is a recommendation, and not a specification, it is believed that the interests of progress in the development of concrete construction will be better served if the best methods known are outlined.

3. On small jobs, or in the more competitive uses of concrete such as building construction, practices somewhat more relaxed than those

*Adopted as a Standard of the American Concrete Institute at its 38th Annual Convention, 1942, as reported by Committee 614; ratified by Letter Ballot May 24, 1942.

here recommended may be economically justified. The profitable degree of such deviation must be determined by those in charge of each job, in view of the conditions which obtain. In general, the Committee has no evidence that good concrete operations cannot be performed as economically as poor ones or that the methods here recommended, where applicable, will result in higher ultimate costs than other methods. The refinements recommended are intended primarily to improve uniformity and to eliminate segregation in aggregates and concrete. Where properly executed, the methods described have resulted in smoother operation and any additional costs resulting from them have been offset by consequent higher production rates.

4. It has been assumed that whoever will give serious consideration to these recommendations will have a reasonable knowledge of the ordinary practices and preparations required in concrete work. For that reason, many routine instructions for measuring, mixing, and placing concrete are omitted. Since the specific objective of these recommendations is maximum uniformity, homogeneity and quality of concrete in place, special consideration is given practices designed to accomplish that end. In order to portray more clearly certain of the principles involved and their application to concreting operations, four figures are included as a part of these recommendations, illustrative of some examples of good and bad practice.

5. For further discussions of the standard aspects of measurement, mixing, and placing, the reader is referred to the 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete and to the particularly excellent and specific document of recommended practice adopted by the American Association of State Highway Officials relating to Measuring, Mixing, and Placing of Concrete for Use in Concrete Pavements. With the recommendations of those reports, ACI Committee 614 is, in general in accord, although a few exceptions in details or in emphasis may be noted.

II. MEASUREMENT

Essential requirement

1. Equipment should be used which is capable of performing accurate measurement. Materials should be handled, and measuring operations performed in such a manner that satisfactory reproducibility of the selected batch assembly and aggregate grading is obtained batch after batch. The objective is ultimate uniformity and homogeneity of the concrete.

Aggregates

2. The number of separate sizes of coarse aggregate which should be used, to reduce segregation in handling to a practical minimum and

to permit reasonably close control of the grading from batch to batch, is dependent upon the maximum size of the material. For particles larger than 1 inch, the ratio of the maximum to the minimum designated sizes should not exceed 2; for coarse aggregate particles finer than 1 inch, this ratio should not exceed 3. The permissible tolerance of undersize or oversize material in each designated size of aggregate should not exceed 10 per cent, and handling operations should be such that variation in these portions of such material is held to a practical minimum.

3. The maximum size of sand should not exceed that which will pass a screen having ⅛-in. openings if it is screened and handled dry. The standard No. 4 sieve (3/16-in. openings) represents a suitable maximum size for wet screened sand which is to remain damp throughout all handling and batching operations, except when the use of the smaller size is desirable to control the percentage of pea gravel (about No. 8 to ⅜-in.) in the mix. When two sizes of sand are used to obtain proper grading, no attempt should be made to blend them by placing alternate amounts in stock piles or in cars and trucks as loaded or by other equally crude procedures. With adequate equipment and ample supervision and inspection, good results are obtainable, when the fine aggregate is produced, by blending the different sizes of sand as they flow into a common stream from regulating gates or feeders. Such methods can, however, produce very irregular results. For the majority of work, separate batching of the fine and coarse sand is preferable since it will produce consistently uniform results under the wide range of common plant and job conditions.

4. The production or purchase of sand should be controlled insofar as possible to minimize variations in grading, particularly in the finer sizes. In plants where sand is washed, care should be exercised to avoid removal of fines essential to good grading and to plastic, workable concrete. For sand passing a ⅛-in. screen, 20 per cent finer than 50 mesh is considered close to an ideal percentage of fines for average concrete work. Lean mixes are benefited by ample percentages of such fines while rich mixes are improved with a smaller percentage of these sizes.

5. Varying amounts of water in aggregates commonly contribute to lack of uniformity. Effort should be made to maintain a uniform moisture content in the aggregate supplied for batching. For this reason aggregates from dry deposits, if not high in absorption, may preferably be prepared and handled dry when washing is not essential. Before placing the material in batcher bins, wet aggregates should be drained until, by actual test, there is little difference in the moisture content of the wettest and driest portions of the materials. Adequate drainage

will require at least 8 hours and additional time may be necessary, depending on the grading of the material and the efficiency of the drainage system provided. On large jobs when thorough drainage is difficult, an electric moisture indicator in the sand batcher and a consistency meter on the mixer will permit rather close control of consistency, even when there are wide fluctuations in aggregate moisture.*

6. Aggregates should be handled in stockpiles and bins in a manner producing minimum segregation of sizes. Stockpiles should be permitted only when built up in layers no thicker than those resulting from truck loads dumped on the same plane. When stockpiling sand and pea gravel, segregation may be further avoided if moisture contents of at least 5 and 2 per cent, respectively, are maintained. Coarse aggregate should be removed from stockpiles, by clamshell bucket or other means, in horizontal layers of such width that aggregate is not permitted to run down the slopes at the edge of the pile. Storage bins should have the smallest practicable equal horizontal dimensions. Round bins are preferable. Bins should have only one outlet located at the center of a bottom sloping to it from all directions at not less than 50 degrees from the horizontal. They should be charged by material falling vertically directly over the outlet. They should be kept as full as possible at all times to avoid breakage and grading changes as the bins are emptied.

7. Reference is made to the sketches, a part of these recommendations, for illustrations of preferred practice in the handling of aggregates. Unless precautions are exercised in the selection and handling of aggregates, refinements in measuring, mixing and placing are nullified.

Batching aggregates

8. *For jobs of from 2,000 to 5,000 cu. yd.*, the cost of hand batching soon justifies a simple weigh batcher installation, into whose bins aggregates may be delivered directly. As the size of the job increases, the saving in materials and the prevention of contamination by dirt or by other sizes adds considerably to the balance in favor of bins as opposed to dumping materials on the ground. As the size of the work increases still greater refinement should be required.

9. *From 2,500 to 10,000 cu. yd. jobs*, cumulative manual weigh batching and from *10,000 to 25,000 cu. yd.*, cumulative automatic weigh batching are recommended. For jobs of over *25,000 cu. yd.*, requirement of individual automatic weighing of the aggregate is justified by the improved perfection of uniformity of measurement. (See favored sketch.) If good inspection and dependable plant men are available,

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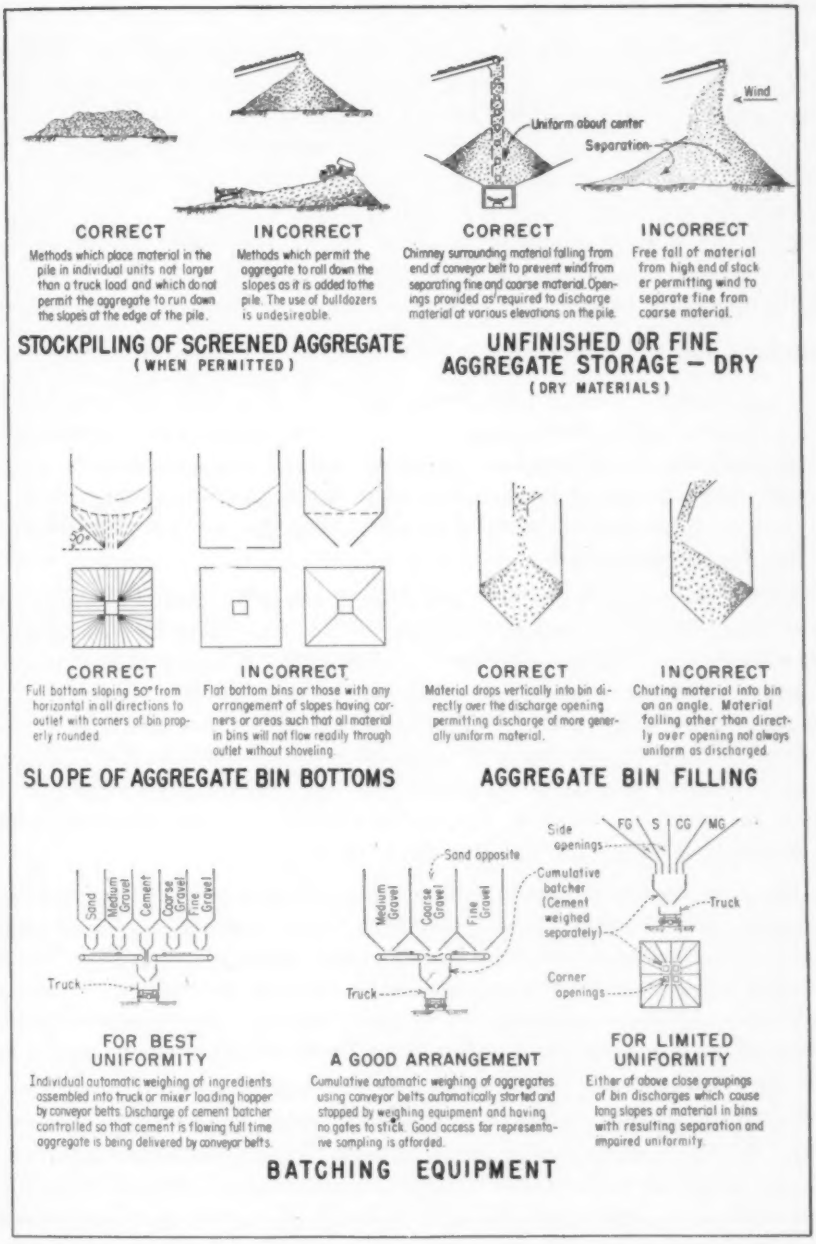
CORRECT
Methods which place material in the pile in individual units not larger than a truck load and which do not permit the aggregate to run down the slopes at the edge of the pile.
INCORRECT
Methods which permit the aggregate to roll down the slopes as it is added to the pile. The use of bulldozers is undesireable.
STOCKPILING OF SCREENED AGGREGATE
(WHEN PERMITTED)
Uniform about center
Separation
Wind
CORRECT
Chimney surrounding material falling from end of conveyor belt to prevent wind from separating fine and coarse material. Openings provided as required to discharge material at various elevations on the pile.
INCORRECT
Free fall of material from high end of stacker permitting wind to separate fine from coarse material.
UNFINISHED OR FINE AGGREGATE STORAGE – DRY
(DRY MATERIALS)
50°
CORRECT
Full bottom sloping 50° from horizontal in all directions to outlet with corners of bin properly rounded.
INCORRECT
Flat bottom bins or those with any arrangement of slopes having corners or areas such that all material in bins will not flow readily through outlet without shoveling.
SLOPE OF AGGREGATE BIN BOTTOMS
CORRECT
Material drops vertically into bin directly over the discharge opening permitting discharge of more generally uniform material.
INCORRECT
Chuting material into bin on an angle. Material falling other than directly over opening not always uniform as discharged.
AGGREGATE BIN FILLING
Sand
Medium Gravel
Cement
Coarse Gravel
Fine Gravel
Truck
FOR BEST UNIFORMITY
Individual automatic weighing of ingredients assembled into truck or mixer loading hopper by conveyor belts. Discharge of cement batcher controlled so that cement is flowing full time aggregate is being delivered by conveyor belts.
Sand opposite
Medium Gravel
Coarse Gravel
Fine Gravel
Truck
A GOOD ARRANGEMENT
Cumulative automatic weighing of aggregates using conveyor belts automatically started and stopped by weighing equipment and having no gates to stick. Good access for representative sampling is afforded.
Side openings
FG
S
CG
MG
Cumulative batcher (Cement weighed separately)
Truck
Corner openings
FOR LIMITED UNIFORMITY
Either of above close groupings of bin discharges which cause long slopes of material in bins with resulting separation and impaired uniformity.
BATCHING EQUIPMENT

Fig. 1—Correct and incorrect handling of aggregates in storing and batching

it may not be economically justifiable to require new equipment, especially for the smaller jobs in the foregoing brackets.

10. Individual automatic and cumulative manual aggregate batchers should be charged through easy operating clamshell or undercut radial type gates. Power-operated gates are used with the automatic batchers and may be used with the manual batchers. Where necessary for the required degree of accuracy, the gate charging the automatic equipment should be arranged to operate with a suitable "dribble" after there is nearly the desired quantity of material in the batcher. Automatic cumulative batchers should preferably be supplied from short conveyor belts operating between the center of the bin bottom and the batcher.

Batching cement

11. Cement for large jobs should be handled in bulk and weighed for each batch, preferably automatically. New equipment for this purpose probably is not justified except on jobs of well over 10,000 cu. yd. Cement should never be weighed with the aggregate. Sacked cement should be measured in units of not less than one sack unless the fractional bags are weighed.

12. Bulk cement batchers should be charged by means of controlled screw conveyors or equally effective devices. Moisture should be thoroughly removed from the air used in jets to loosen the cement in storage. Cement hoppers and batchers should have smooth metal or metal covered sides with rounded corners. Silos, bins, and hoppers for cement should be governed by principles outlined above for aggregate bins. Control should be available on the batcher discharge gate so that, in event of over-run, the excess cement may be withheld unless it is removed before the batch is discharged.

13. Modern specifications issued by government and highway agencies contain excellently detailed specifications for manual and automatic weigh batching equipment for cement and aggregate. Ordinarily, the present equipment on the market, when in good mechanical condition, will have approved tolerances in weighing ability. Operation should be required within a degree of accuracy of 1 per cent for cement and 2 per cent for aggregate.

14. Bulk cement is often lost or scattered indescriminately in dropping from the batcher to trucks or batch cars below. Free fall of cement should never be permitted. A typical good method of preventing this is to enclose the discharging cement in a narrow canvas boot long enough so that its outlet may be buried in the cement when loading portable batch compartments. Such compartments should contain a separate section for the cement, attached to and operating with the individual

batch release gate. The boot may be used as a tremie or the cement may be eased into the compartment by controlling the amount of kinking or doubling back of the canvas at the lower end of the boot. The canvas may then be lifted, shaken, and dropped into the next compartment. A telescopic rubber hose drop-chute from the cement batcher is also available for this purpose. If a separate section is not provided for the cement in each batch compartment, or the cement is not enclosed in the aggregate, tarpaulin covers should be required during transportation. For placing bulk cement into a batch hopper for a stationary mixer, a pipe of suitable size to hold the cement should extend from the batcher discharge to a level near the bottom of the hopper. After covering the end of this with the aggregate, the cement may be discharged. It will then enter the mixer without loss of dust, and it will be well distributed through the entering aggregate.

Water measurement

15. Water measurement on the larger jobs justifies the recommendation that for pavers and other portable mixers, there be used only automatic meters of proved mechanical dependability, or automatic measuring tanks of the vertical cylinder center-siphon discharge type, capable of routine measurement within an accuracy of 1 per cent under all operating conditions. For central mixing or batching plants, either of these devices or automatic weigh batchers of proved dependability and accuracy within 1 per cent should be required. Tanks should be permitted only as an auxiliary part of the automatic meter or weigh batcher, but not as a means of measurement, except in the form of a vertical cylinder with center-siphon discharge.

The small jobs

16. For the small job, usually less than two to five thousand cubic yards, concreting operations are likely to be so irregular that many of the foregoing preludes to uniformity in the concrete, as far as aggregate preparation and handling are concerned, are often next to impossible of attainment. It is well to keep them in mind, however, as standards to be used wherever possible. Cement should be handled and measured in sacks. Mixers should be of sufficient capacity and batches of such a size that no fractional bags of cement are required unless they are weighed for each batch. Water measuring equipment of expensive refinement may not be economical for the small job, but the use of simple equipment that is accurate, positive and dependable is justified and should be required. A manually operated standard disc water meter, of capacity according to the size of the mixer, equipped with a vertical-face, set-back register is not expensive and is recommended where water temperatures do not exceed 100°F. A vertical cylinder tank with center-

siphon discharge is equally satisfactory, and should be used if water temperatures are greater than 100°F. Aggregates should be weighed. In a metropolitan area where weigh batched aggregates or plant controlled ready mixed concrete is available, one of these should be used, even at a reasonable premium, in preference to handling and measuring the materials on the job. Where such services are not available, the quantities for each batch should be weighed in wheelbarrows or buggies on platform scales now generally available for this purpose on small jobs.

General

17. If standards of uniformity, such as are readily obtainable with modern batching equipment, are to be maintained uniformly throughout an operation, there is greater need for emphasis of correct procedure in incidental operations than on the actual weighing of ingredients. Care should be exercised that batches accurately assembled arrive as uniform batches in the mixer. These sources of error should be avoided: (1) Over-lap of batches in loading and discharging multiple batch trucks and cars. (2) Loss of material in transferring batches to the skips of portable mixers. (3) Loss or "hang up" of a portion of one batch, or its inclusion with another, when dry batches are transferred by means of belts and hoppers. All such damage to the correctness of the quantities in each assembled batch should be strictly avoided and methods should be approved or discarded accordingly.

III. MIXING

Essential requirements

1. Thorough mixing is, of course, a first essential. Equipment and methods used shall be such as to insure uniformity of strength, and uniformity of consistency, cement and water content, and aggregate grading from beginning to end of each batch *as discharged*. For concrete work of the highest general quality, mixing equipment should be used which is capable of handling concrete suited to the particular work, from the standpoint of its placeability and workability *in the forms* (preferably by means of vibration where this is applicable). Mixing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water than indicated for best results on the basis mentioned is not recommended.

The mixer

2. The mixer should preferably have a combination of blade arrangement and drum shape such as to insure an end to end exchange of the materials parallel to the axis of rotation as well as a rolling, folding, or spreading movement of the mix over on itself as the batch is mixed. Otherwise the action should be such that comparable mixing efficiency

and uniformity of the batch is obtained within a reasonable mixing period. The mixer should be operated at the speed designated by the manufacturer, unless a reasonable change of speed definitely shows better results.

3. Any mixer leaking mortar or causing waste of materials due to faulty size, shape, or operation of charging equipment should be taken out of service immediately until satisfactory repairs and improvements have been made to eliminate such waste. Hardened concrete or mortar should not be permitted to accumulate on the inner surfaces of mixing equipment. Worn blades should not be continued in service when they materially decrease mixing efficiency.

Size of batch

4. The size of the batch in job mixers should not exceed the manufacturer's guaranteed capacity of 10 per cent more than the rated capacity of the mixer. The rated capacity should be exceeded only when: (1) it results in no loss of mortar during mixing, and (2) tests show the larger batch to be properly mixed. When, to shorten time of discharge or to reduce segregation due to retention of coarse aggregate, it is desired to retain a portion of the batch in excess of the normal "hold back," the amount of material held back should be taken into account in fixing the size of the batch in relation to the capacity of the mixer.

The charging operation

5. All ingredients of the mix should enter the mixer promptly and as uniformly as possible at the same time. It is particularly important in charging stationary mixers that the solid materials be arranged in the charging hopper in such a manner that no one of them enters separately but that proportional amounts of each (particularly of cement and sand) will be in all parts of the stream of material as it flows into the mixer. To avoid "gumming" and for greater uniformity, cement should never be charged separately, especially ahead of the aggregates but should be enclosed within the other materials. Water for the batch should be released first and continue to flow while the solids are entering the mixer, and should have completed flowing shortly after the last of the batch has entered the drum. This flow should not continue for more than the first 25 per cent of the mixing time, and should be stopped as much short of this as the apparent requirements of good operation of the particular mixer will permit. Where meters are used, and particularly if water pressure is low, auxiliary tanks, into which the quantity of water for the batch may be measured in advance, equipped with a discharge pipe to the mixer of such a size as to insure the foregoing rate of delivery, may be necessary.

Mixing time

6. The net mixing time for a one-yard or smaller mixer when mixing gravel concrete of medium consistency containing 1.5 bbl. or more of cement per cu. yd. should not be less than 1 minute under the best conditions. For concretes containing less cement, mixed to drier consistencies, or containing an unusually harsh mixing aggregate, time should be increased 25 per cent to 50 per cent according to the degree of departure from concretes of the most readily mixed type.

7. For job mixers larger than one cu. yd. capacity, particularly when mixing mass concrete, a nominal minimum net mixing time of 1 minute plus ¼ minute for each additional cubic yard of capacity should be required. The use of mixers unable to pass subsequently prescribed tests for mixing efficiency after twice this period of mixing and when charged indescriminately, should be prohibited. The total net mixing time for mixers meeting this test should be the foregoing minimum (determined by the size of the mixer) plus such additional time as necessary to bring the batch within the requirements of the mixer efficiency test when the mixer is charged and operated in accordance with practice which may be established as routine for the job.

8. The mixing period should be measured from the time when all the solid materials are in the mixer drum provided that all the mixing water shall be introduced before 25 per cent of the mixing time has elapsed. Over-mixing of more than three times the above required periods should not be permitted and for that reason all mixing equipment should be so arranged that it can be stopped and started under full load.

9. A batch timer and counter, including an automatic lock which will release the discharge lever only at the end of the proper mixing period, and an audible indicator, should be installed and maintained in operation on each concrete mixer. On truck mixers the audible indicator is unnecessary. For jobs requiring 2 cu. yd. or larger stationary mixers a recording watt meter or consistency meter has definite advantages and should be required on each mixer because it is more reliable as a batch counter, and because it provides a permanent record of number of batches, type of batch, mixing time, retempering, and the uniformity of consistency. Failure of any of this equipment should be sufficient cause for discontinuance of the use of the mixer until the device is repaired or a new one is installed.

Retempering

10. Retempering or indiscriminate addition of water to delayed batches which have become stiffer than the usual consistency should be

prohibited. Under careful supervision an increment of water may sometimes be added with results beneficial to essential workability, provided the maximum allowable water-cement ratio is not exceeded. Concrete delayed in placing should not be wasted unless the amount of additional water, necessary to restore sufficient workability to permit it to be placed, would exceed the allowable water-cement ratio. Batches considerably wetter than the specified consistency, or batches in which there has been some obvious error in the measurement of solid ingredients, particularly a shortage of cement, should be wasted unless proper corrections can be made at once and the corrected batch thoroughly mixed.

Ready mixed concrete

11. The use of ready mixed concrete is an important development in connection with general concrete construction—particularly in metropolitan areas. A ready mixed concrete operation, adequately equipped and supervised, affords excellent opportunities for the control of concrete quality. Thus, ready mixed concrete may be made to offer the small job the control facilities ordinarily available only to large jobs.

12. Ready mixed concrete may be centrally mixed, truck mixed (mixed in transit), or partially centrally mixed and partially truck mixed (often referred to as "shrink-mixing"). The use of ready mixed concrete introduces certain new problems of control to which, while not fundamentally different from those encountered in other forms of concreting operations, attention should be directed.

13. Centrally mixed concrete presents about the same problems as job-mixed concrete which must be transported for considerable distances from the plant to the place of deposit. The possibilities of loss in consistency, resulting from lapse of time between mixing and placing, should be given consideration. For normal periods and temperature, and when agitator trucks are used for transportation, this factor is generally negligible. Nevertheless, precautions should be taken to minimize loss of slump by expediting delivery, eliminating delays and, in warm weather, keeping the concrete as cool as practicable by using cold mixing water, avoiding hot cement, and by shading and sprinkling aggregate. Under conditions where loss of workability cannot be otherwise controlled and is likely to be a serious matter, mixing at the forms or adequately controlled truck mixing as hereinafter outlined, will eliminate the difficulty.

14. In the case of truck mixing operations it is necessary to exercise more than usual control of all factors governing consistency because of the difficulty of judging consistency until the concrete is delivered and discharge is started. Variations in consistency may be minimized by:

(a) Handling aggregates in such a manner as to reduce variations in grading and moisture content to the greatest practical extent.

(b) Exercising accurate control over the quantity of mixing water which may be admitted to the truck mixers.

(c) Telephone communication between the point of placing and the batching plant to keep minimum the quantity of concrete arriving before a change order goes into effect.

(d) Responsible technical supervision of the entire operation.

15. In some cases satisfactory control may be readily obtained only by withholding the final increment of the mixing water until the mixer arrives at the form and then doing the additional mixing as necessary to incorporate thoroughly the added water. As pointed out earlier, complete truck mixing en route will produce consistencies of satisfactory uniformity, with minimum water content, only when special care is exercised in the control of grading and moisture content; in the absence of such control resort should be made to mixing at the forms or to completing the mixing at the forms. While every effort should be made to have concrete batches arrive at the job at a suitable consistency, it is possible to make adjustments after arrival if proper provisions are made in specifications and on the job. If the batch arrives too dry, but contains less water than the maximum allowed, additional mixing water, within specification limits, may be added and additional mixing done at the job site. In general, batches which arrive too dry, and already contain the maximum allowable mixing water, or batches which arrive too wet, should be rejected. However, adjustments may be made where conditions permit, although they are to be avoided since they interfere with orderly routine. Every care should be exercised to see that concrete arrives at the job with the desired consistency.

16. Truck mixers should be operated at a mixing speed in accordance with the recommendations of their manufacturer and should also be capable of operating at a suitable lower agitating speed of not less than 2 nor more than 6 r.p.m. of the drum. The time of mixing in truck mixers should be not less than 50 revolutions of the drum or blades at the rate of rotation specified by the manufacturer as mixing speed. The volume of the batch should not exceed the manufacturer's rating of the truck mixer's capacity and, in any event, should not be greater than fifty-seven and one-half (57.5) per cent of the gross volume of the drum or container. If, from any cause, this amount of mixing is insufficient, as determined by the subsequent tests for mixer efficiency or by other means, additional mixing should be required until satisfactory results are obtained. In truck mixing and shrink mixing, not more than 150

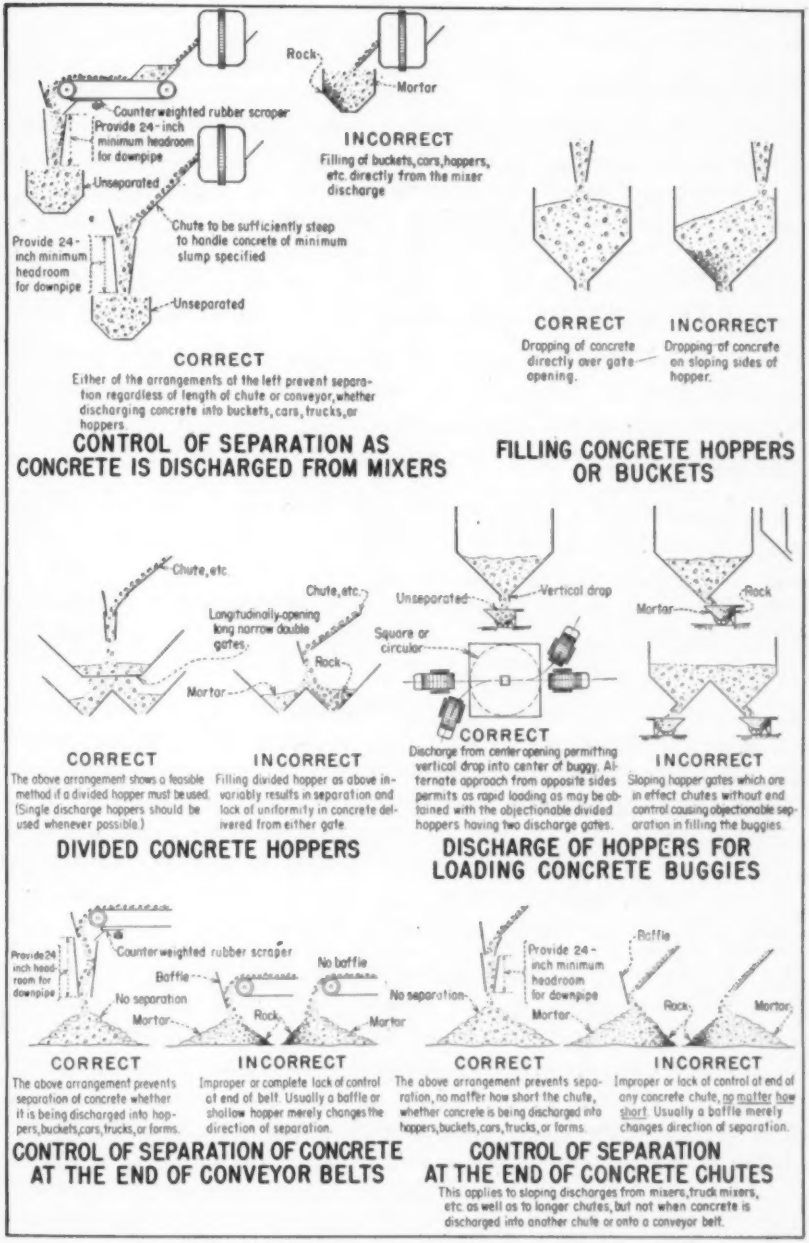
Rock
Mortar
Counter weighted rubber scraper
Provide 24-inch minimum headroom for downpipe
Unseparated
Chute to be sufficiently steep to handle concrete of minimum slump specified
Provide 24-inch minimum headroom for downpipe
Unseparated
INCORRECT
Filling of buckets, cars, hoppers, etc. directly from the mixer discharge
CORRECT
Either of the arrangements at the left prevent separation regardless of length of chute or conveyor, whether discharging concrete into buckets, cars, trucks, or hoppers.
CONTROL OF SEPARATION AS CONCRETE IS DISCHARGED FROM MIXERS
CORRECT
Dropping of concrete directly over gate opening.
INCORRECT
Dropping of concrete on sloping sides of hopper.
FILLING CONCRETE HOPPERS OR BUCKETS
Chute, etc.
Chute, etc.
Longitudinally-opening long narrow double gates
Rock
Mortar
CORRECT
The above arrangement shows a feasible method if a divided hopper must be used. (Single discharge hoppers should be used whenever possible.)
INCORRECT
Filling divided hopper as above invariably results in separation and lack of uniformity in concrete delivered from either gate.
DIVIDED CONCRETE HOPPERS
Unseparated
Vertical drop
Square or circular
CORRECT
Discharge from center opening permitting vertical drop into center of buggy. Alternate approach from opposite sides permits as rapid loading as may be obtained with the objectionable divided hoppers having two discharge gates.
Mortar
Rock
INCORRECT
Sloping hopper gates which are in effect chutes without end control causing objectionable separation in filling the buggies.
DISCHARGE OF HOPPERS FOR LOADING CONCRETE BUGGIES
Provide 24 inch headroom for downpipe
Counter weighted rubber scraper
Baffle
No separation
Mortar
Rock
No baffle
Mortar
CORRECT
The above arrangement prevents separation of concrete whether it is being discharged into hoppers, buckets, cars, trucks, or forms.
INCORRECT
Improper or complete lack of control at end of belt. Usually a baffle or shallow hopper merely changes the direction of separation.
CONTROL OF SEPARATION OF CONCRETE AT THE END OF CONVEYOR BELTS
No separation
Provide 24-inch minimum headroom for downpipe
Mortar
Baffle
Rock
Mortar
CORRECT
The above arrangement prevents separation, no matter how short the chute, whether concrete is being discharged into hoppers, buckets, cars, trucks, or forms.
INCORRECT
Improper or lack of control at end of any concrete chute, no matter how short. Usually a baffle merely changes direction of separation.
CONTROL OF SEPARATION AT THE END OF CONCRETE CHUTES
This applies to sloping discharges from mixers, truck mixers, etc. as well as to longer chutes, but not when concrete is discharged into another chute or onto a conveyor belt.

Fig. 2—Correct and incorrect methods of handling mixed concrete

revolutions of the drum should be at a peripheral speed in excess of that designated as agitator speed. Additional mixing, if necessary, should be at agitator speed.

Measure of mixing efficiency

17. The efficiency of the mixing operation in intermingling the constituents of the concrete in all types of mixers should be such that at the end of the prescribed or permitted mixing period spread in the water-to-fines ratios, W/F, of three 1500-gram samples of mortar from well separated localities in the mixer, should be within ten per cent of the average of the three samples; W being the mixing water and F being the cement and other fine material passing the No. 100 screen.*

18. The ratio of coarse aggregate to mortar should appear to be uniform in all parts of the mixer on visual inspection and this should be borne out by subsequent observation of the workability of the concrete.

The discharge operation

19. No portion of the time required for discharging any type of mixer should be considered a part of the required net mixing time. The discharge facilities of all types of mixers should be capable of ready discharge of concrete of the stiffest consistency which should be placed by means of vibration. To meet this requirement, all mixers should be capable of ready discharge of concrete of 1 inch slump.

20. To preserve the uniformity of distribution of materials and the usual homogeneity of the concrete in the mixer immediately prior to discharge, all types of mixers discharging into hoppers, buckets, cars, etc., should be so equipped that the concrete will drop vertically, not diagonally, into such containers in accordance with the principles shown in the sketches which constitute a part of these recommendations. Separation of coarse aggregate from the mortar, commonly resulting as concrete is discharged from the mixer, is thus avoided.

21. The blade arrangement and discharge mechanism of all types of mixers, including agitating, shrink, and truck mixers, should be such that the amount of aggregate larger than $\frac{3}{4}$ inch in any portion of the batch will not differ by more than 20 per cent from the amount of such aggregate in any other portion of the batch. This may be determined readily by comparing the weights of coarse aggregate retained when at least 200 lb.† of concrete from each portion of the batch in question is washed over a $\frac{3}{4}$-in. screen. Until or unless equipment is available which will meet this requirement, and is operated in this manner, when

*For one method of test see "Mortar Test for Mixer Efficiency," Method B, U. S. Bureau of Reclamation Concrete Manual, third edition, January 1941, p. 419.

†A 500-lb. sample is preferable if it does not contain portions of the batch other than those supposedly represented.

the fraction of the batch containing excessive amounts of coarse aggregate is ordinarily the last portion discharged, that fraction should be withheld from discharge and mixed with the succeeding batch. As heretofore recommended in this case, the size of batches charged into the mixer should be reduced by this amount. In some truck mixers this type of separation may be avoided by reversing the direction of rotation for one or two revolutions prior to the final discharge.

General

22. In general, all of the foregoing recommendations should be considered where applicable in connection with the mixers of proper capacity for the job regardless of the size of the job or its classification.

IV. PLACING

Essential requirements

1. Only those methods and arrangements of equipment should be used which will reduce to a minimum the separation of coarse aggregate from the mortar at all points from the mixer to the forms. For highest quality of concrete *in place*, the placing equipment should be considered for its ability to handle mixes ideal for the particular work, from the standpoint of the concrete placeability and workability *in the forms* (preferably by means of vibration where this is applicable). Equipment should be capable of expeditious placing of concrete of the proper consistency, grading and maximum size of aggregate, at the rate most advantageous to good workmanship. Selection or approval of any part of placing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water, or smaller coarse aggregates, than indicated for best results on the above basis, is not recommended.

Avoidance of separation

2. The most important consideration in handling and placing concrete is the avoidance of separation of coarse aggregate from the concrete. Particular attention must be paid to that tendency at ends of chutes and conveyor belts, at hopper gates and at all other points of discharge, if uniformity and homogeneity of concrete in place and good workmanship are to be assured. No consideration should be given to the common fallacy that separation occurring in handling will be eliminated in the course of other operations. Separation should be prevented—not corrected after its occurrence.

3. Important in arranging equipment to prevent separation is the provision that the concrete shall *drop vertically* into the center of whatever container receives it. The importance of this increases very greatly with increases in slumps, maximum size, amounts of coarse aggregate,

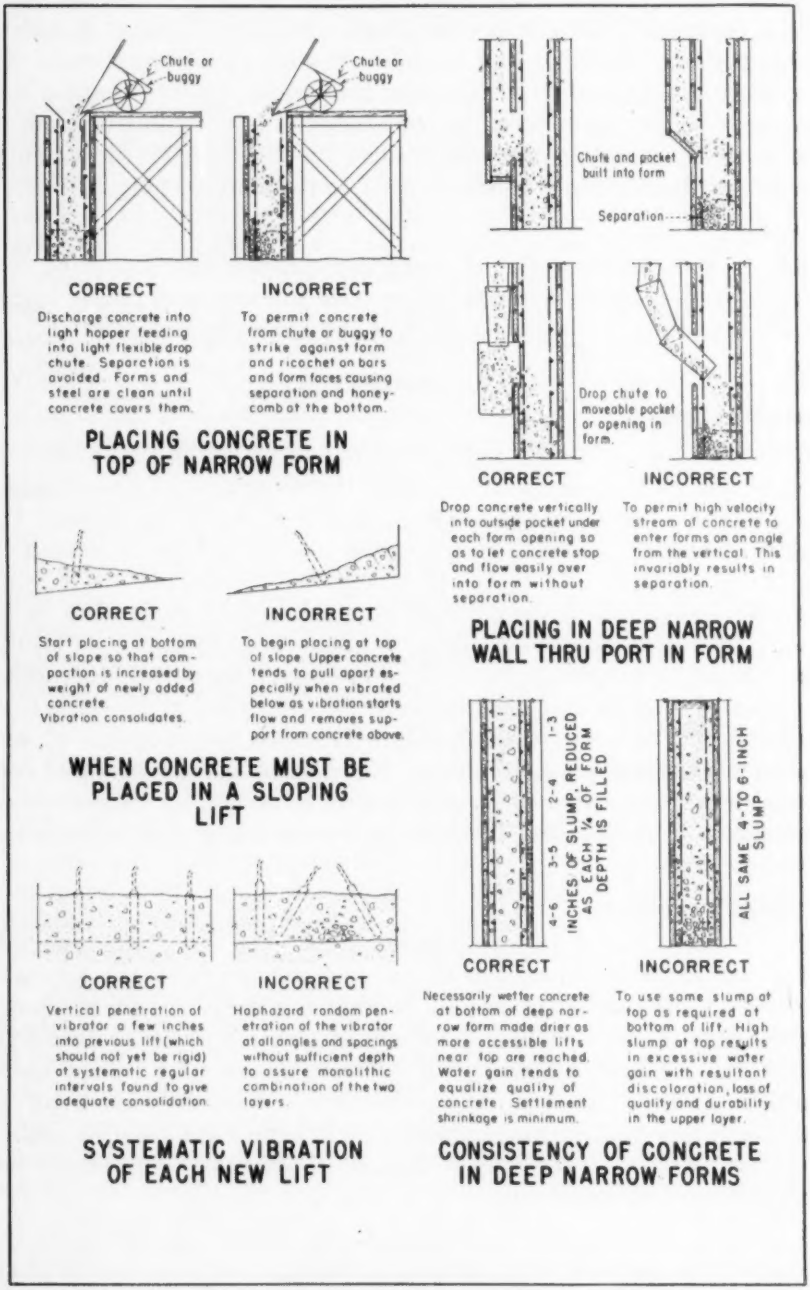
Chute or buggy
Chute or buggy
CORRECT
INCORRECT
Discharge concrete into light hopper feeding into light flexible drop chute. Separation is avoided. Forms and steel are clean until concrete covers them.
To permit concrete from chute or buggy to strike against form and ricochet on bars and form faces causing separation and honeycomb at the bottom.
PLACING CONCRETE IN TOP OF NARROW FORM
CORRECT
INCORRECT
Start placing at bottom of slope so that compaction is increased by weight of newly added concrete. Vibration consolidates.
To begin placing at top of slope. Upper concrete tends to pull apart especially when vibrated below as vibration starts flow and removes support from concrete above.
WHEN CONCRETE MUST BE PLACED IN A SLOPING LIFT
CORRECT
INCORRECT
Vertical penetration of vibrator a few inches into previous lift (which should not yet be rigid) at systematic regular intervals found to give adequate consolidation.
Haphazard random penetration of the vibrator at all angles and spacings without sufficient depth to assure monolithic combination of the two layers.
SYSTEMATIC VIBRATION OF EACH NEW LIFT
Chute and pocket built into form
Separation
Drop chute to moveable pocket or opening in form.
CORRECT
INCORRECT
Drop concrete vertically into outside pocket under each form opening so as to let concrete stop and flow easily over into form without separation.
To permit high velocity stream of concrete to enter forms on an angle from the vertical. This invariably results in separation.
PLACING IN DEEP NARROW WALL THRU PORT IN FORM
4-6 3-5 2-4 1-3 INCHES OF SLUMP, REDUCED AS EACH ¼ OF FORM DEPTH IS FILLED
ALL SAME 4-TO 6-INCH SLUMP
CORRECT
INCORRECT
Necessarily wetter concrete at bottom of deep narrow form made drier as more accessible lifts near top are reached. Water gain tends to equalize quality of concrete. Settlement shrinkage is minimum.
To use same slump at top as required at bottom of lift. High slump at top results in excessive water gain with resultant discoloration, loss of quality and durability in the upper layer.
CONSISTENCY OF CONCRETE IN DEEP NARROW FORMS

Fig. 3—Correct and incorrect methods of placing concrete.

and with reductions in cement content. Falling concrete should be closely confined in a down pipe of the proper size to within a few feet of the place of deposit in the forms or other container *and the final drop must be vertical* if separation is to be prevented. Application of this principle to various common details of concrete handling equipment is shown in the accompanying sketches.

4. Where applicable, the use of bottom discharge buckets is a superior method of handling and placing mass concrete, provided: (1) they are of a size, and may be discharged in a manner and with such frequency, that the concrete may be placed in approximately horizontal layers while the previous layer is still soft; (2) they are capable of discharging concrete of the stiffest consistency specified; (3) successive batches are so placed as to afford opportunity for thoroughly working the concrete by means of internal vibrators; (4) only complete mixer batches are placed in the bucket; and (5) separation is avoided in filling the buckets (see sketch).

5. Concrete should not be dropped through reinforcement steel, or into any deep form whether reinforcement is present or not, so as to cause separation of the coarse aggregate from the mortar by repeatedly hitting rods or the sides of the form as it falls. For placing under such conditions, hoppers and, if possible, vertical ducts should be used in the forms, or other means employed so that the concrete may reach the place of final deposit without separation or coating the steel and forms with mortar that will dry out long before it can be covered with concrete. In difficult cases of deposit of this kind in deep, narrow, reinforced walls, where even narrow rectangular ducts can not be inserted, good results may be obtained by closing each third space between studs on one side of the wall for a duct through which to drop the concrete which then enters the form through holes cut in the sheathing at vertical intervals not greater than 4 ft. as the concrete rises to each opening. A pocket should be provided at the bottom of each duct successively, below each opening, so that the concrete will stop and flow easily over into the forms with minimum scattering and separation. A good internal vibrator should be in operation on each side of an opening while concrete is entering the forms.

Vibration

6. Except for a few special sections, such as thin slabs, no method of compaction and consolidation of concrete in place is superior to effective internal vibration. Vibration contributes general improvement to concrete work in the fact that superior workmanship may be obtained at lower cost, and because it permits successful routine use of a concrete

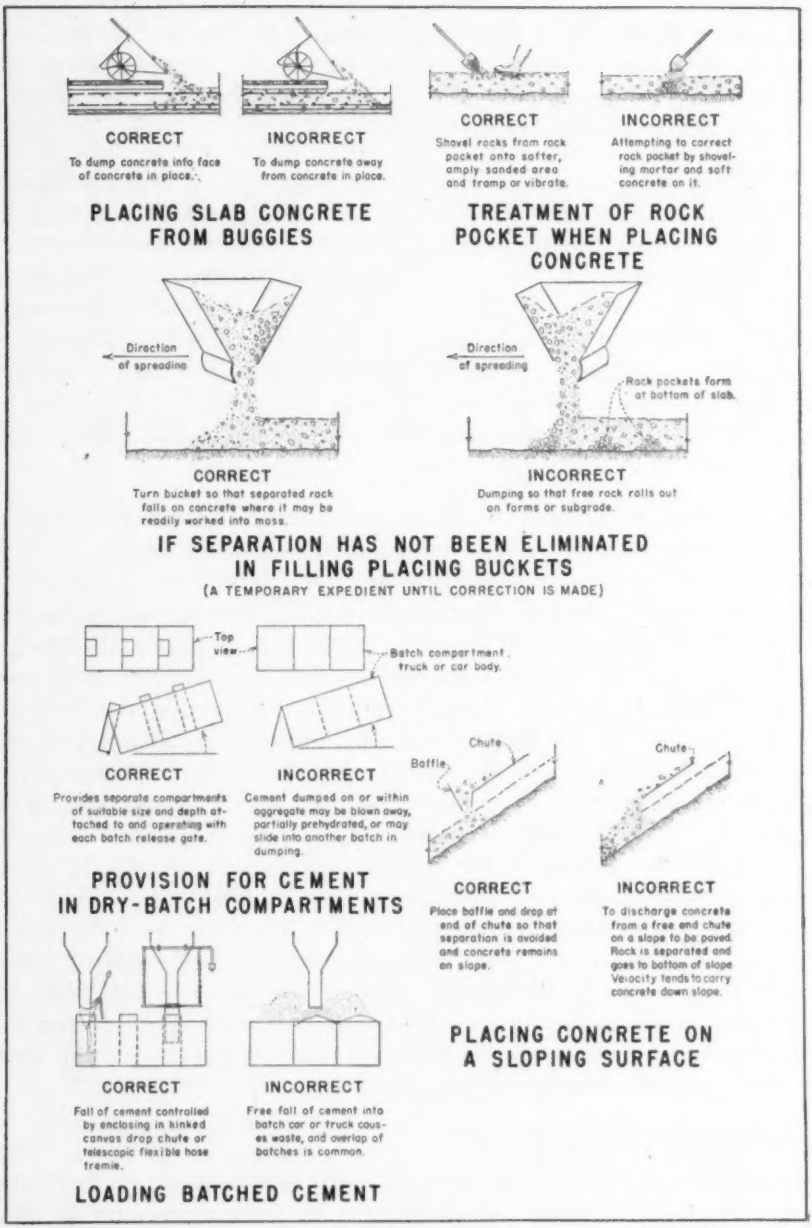
CORRECT
To dump concrete into face of concrete in place.
INCORRECT
To dump concrete away from concrete in place.
PLACING SLAB CONCRETE FROM BUGGIES
CORRECT
Shovel rocks from rock pocket onto softer, amply sanded area and tramp or vibrate.
INCORRECT
Attempting to correct rock pocket by shoveling mortar and soft concrete on it.
TREATMENT OF ROCK POCKET WHEN PLACING CONCRETE
Direction of spreading
CORRECT
Turn bucket so that separated rock falls on concrete where it may be readily worked into mass.
Direction of spreading
Rock pockets form at bottom of slab.
INCORRECT
Dumping so that free rock rolls out on forms or subgrade.
IF SEPARATION HAS NOT BEEN ELIMINATED IN FILLING PLACING BUCKETS
(A TEMPORARY EXPEDIENT UNTIL CORRECTION IS MADE)
Top view
Batch compartment, truck or car body.
CORRECT
Provides separate compartments of suitable size and depth attached to and operating with each batch release gate.
INCORRECT
Cement dumped on or within aggregate may be blown away, partially prehydrated, or may slide into another batch in dumping.
PROVISION FOR CEMENT IN DRY-BATCH COMPARTMENTS
CORRECT
Fall of cement controlled by enclosing in kinked canvas drop chute or telescopic flexible hose tremie.
INCORRECT
Free fall of cement into batch car or truck causes waste, and overlap of batches is common.
LOADING BATCHED CEMENT
Chute
Baffle
CORRECT
Place baffle and drop at end of chute so that separation is avoided and concrete remains on slope.
Chute
INCORRECT
To discharge concrete from a free end chute on a slope to be paved. Rock is separated and goes to bottom of slope. Velocity tends to carry concrete down slope.
PLACING CONCRETE ON A SLOPING SURFACE

Fig. 4—Correct and incorrect methods in handling materials and placing concrete

less wet than the usual "medium" consistency, resulting in more durable, higher quality concrete. The advantages and importance of vibration where applicable are now so well established that it is worthy of serious favorable consideration for small jobs of a few hundred yards, and its requirement is recommended for all larger concrete work.

7. Detailed recommendations are omitted here and the reader is referred to "Recommendations for Placing Concrete by Vibration" reported by ACI Committee 609, ACI JOURNAL, Mar.-Apr. 1936; *Proceedings* V. 32, p. 445. Sufficient to say that the name of vibration is not enough. Equipment should be powerful, of high frequency, efficient, and rugged. Operators should be experienced, competent, dependable and energetic. Ample standby units and parts should be provided. Care should be exercised that concrete is not over-vibrated, particularly if it exceeds 4 in. in slump. Vibrators should not be used improperly to "transport" concrete in the forms. Vibrators should be inserted and withdrawn at many points, from 18 in. to 30 in. apart, for short periods, (usually from 5 to 15 seconds is sufficient) in preference to insertion for longer periods at wider intervals. Systematic spacing of insertions of the vibrator should be established to insure that no concrete remains unvibrated.

8. Inadvertent or intended revibration of concrete or steel embedded in it, any time before the concrete becomes so far set that it will not again become plastic by continued vibration, is not detrimental but may actually increase the strength of the concrete and its bond with the steel. As far as the ordinary job is concerned there is little likelihood of damage from revibration of lower lifts or from vibration transmitted by the steel, provided there is avoidance of actual rupture of newly hardened concrete that will not again become plastic by the vibration. This condition can be determined by an experienced operator from the action and sound of the vibrator.

9. In some cases, vibration may be advantageously supplemented by spading at the face of the forms. The use of flattened spading tools or rods (together with concrete placed in shallow lifts) is helpful in liberating air bubbles which often deface vertical walls and invariably are found on surfaces molded by forms which slope toward the concrete. Under sloping forms vibration without spading tends to aggravate the characteristic imperfections usually found on such surfaces and overvibration is particularly undesirable. (It is in the improvement of such surfaces that absorptive form lining is finding a very valuable application.) At corners, obstructions, and other points where perfect placing may be in question, supplementary hand rodding of the concrete is desirable.

Provision for handling concrete of the proper consistency

10. Objection is frequently made on the job to a relative stiff consistency which will not flow down a certain chute, drop out of a certain hopper or discharge through certain gates, although it is freely admitted that it is readily workable in the forms, particularly when properly vibrated. Such an objection is not valid and should not be sustained if the drier consistency has been determined in advance to be of practicable workability in the forms and has been made a requirement of the specifications. It is the function of concrete handling and placing equipment to handle and place concrete of proportions and consistencies that can be properly worked in the forms and selected for the results required in the forms. Limitations on consistencies and proportions should not be imposed by inadequate chutes, hoppers or gates.

11. Thus it is important, in the design and approval of concrete handling equipment that chutes, where necessary, be amply steep, metal or metal lined, round bottomed, of large size, rigid, and protected from overflow. Discharge gates and hoppers should be large enough to pass quickly and freely concrete of the stiffest consistency likely to be found practicable for placing in the forms by means of vibration. In many cases chutes steeper than 2 to 1 should be used, and double or triple the usual area of hopper outlets and gates would not be excessive.

12. Except where loss of slump due to delayed placing is unimportant and no increase in the original amount of mixing water is made because of it, or when remixing without additional water after a period of delay is desirable and practicable for reduction of shrinkage due to setting, every effort should be made to keep as short as possible the time elapsed from the moment water and cement come together, and the arrival of the concrete in the forms. This facilitates control of uniformity of consistency of the concrete in the forms, and reduces to a minimum the water content of the concrete, and the variable loss of slump between the mixer and the forms. For this same reason concrete should not be exposed in thin streams in long chutes or on long conveyor belts in which condition it is subject to the variable effects of all kinds of weather. It should not be necessary to mix the concrete any wetter than it may be worked in the forms by means of vibration.

V. GENERAL CONSIDERATIONS

1. In mixing and placing concrete, all concerned should remain aware that shrinkage cracks and lack of durability are primarily proportional to the volume of mixing water per unit volume of concrete within the range of practical and suitable mixes for most work. Though in many

cases they are, they *may* not be correlative to the water-cement ratio except for mixes otherwise identical in aggregate and cement content, or for mixes identical only in water content and consistency. An example is the case in which both cement and water are added without changing the water-cement ratio but with an increase in slump. Another, when increases in cement and water are accompanied by use of finer grading of aggregate resulting in little change in consistency. Such variations in unit water content proportionately affect the ultimate quality of the concrete without relation to the water-cement ratio. The more a form is filled with the right combination of solids, and the less it is filled with water, the better will be the resulting concrete. Thus to this end, moderation in the use of water, cement, and fine aggregate, together with the use of aggregate graded to the largest practical maximum size, all in conformity with the elemental needs of the work, should be consistently practiced. Only as much cement should be used as is required to obtain adequate strength and other essential properties; and only sufficient water and fine aggregate should be used as is required to obtain concrete with no more than a degree of workability which may be properly worked in the forms by means of vibration or by more appropriate methods where vibration is not applicable.

2. Sufficient mixing and placing capacity (with judgment in planning the progress of the work) should be provided so that the work may be kept alive and free from "cold joints." In all formed structures, including tunnel linings, concrete should be placed in horizontal layers of not greater depth than 2 feet, particularly avoiding inclined layers and inclined construction joints. It is very important that each layer be placed while the previous layer is still soft. Concrete should not be allowed or caused to flow horizontally or on slopes in the forms. Where concrete must be placed on a slight slope, placing should begin at the lower end of the slope and progress upward thereby increasing the compaction of the concrete. Pneumatic means of placing concrete should, in general, be avoided wherever an alternative method may be used.

Construction joints

3. For the sake of appearance, irregular construction joints should not be permitted. Each lift of concrete should be filled up to a temporary grade strip, or preferably a "V" or a slightly beveled rectangular strip should be left on the forms at the line of the joint where such grooves can be located so as not to detract from the appearance of the work and so as to be less unsightly than the usual construction joint. Particular precautions should be taken to secure the forms tightly against the concrete at the joint, since, otherwise, an unsightly offset will occur

and mortar from the subsequently placed concrete will disfigure the surface of the concrete in place.

4. The surface of the construction joint should be prepared in a manner that will insure bonding with the concrete later placed on it, and if the joint must be watertight the preparation of the joint must be particularly thorough. It is not difficult to obtain a good joint where good quality low-slump concrete has been used at the top of the lift and has not been overworked. Where wet consistencies have been used, or where excessive vibrating or working has brought water or mortar to the surface, the concrete at the surface of the lift is usually so inferior that it is not easy to obtain a good joint.

5. In the latter case the so-called initial jet cleanup is a good expedient for such an inferior joint condition, which on good work would not be encountered. The first step should be the removal of all laitance and inferior surface concrete and the washing of mortar from protruding aggregate by means of a strong jet of air and water, at approximately 100 psi pressure, after the concrete has hardened sufficiently to prevent the jet from raveling the surface below the desired depth and from forming cloudy pools of water that will leave a film on the surface when they dry. The surface of joints so treated should be especially well moist-cured, preferably by a 1-inch layer of saturated sand, and, if possible the surfaces should never be permitted to dry out during the interval before concrete is placed on it. Before placing the new concrete the surface should be restored to the bright clean condition existing immediately following the initial jetting by means of vigorous brushing with fine wire hand brushes or by sandblasting as necessary. If the surface has been properly wet sand cured, very little final scrubbing or sandblasting will be necessary.

6. Initial jetting, followed by a thorough final cleanup can also be used with good results where quality of concrete at the joint surface is good. It produces no better results, however, than can be readily obtained, on surfaces which have been properly placed at the right consistency, by omitting any initial treatment and removing the surface film and dirt to a bright new surface by sandblasting and washing immediately prior to placing the new concrete. The initial jetting method including a comparable final cleanup is usually found to be more expensive than the final sandblasting method and is not as foolproof. Final sandblasting without initial cleanup has been found to produce excellent results economically on horizontal joint surfaces of mass concrete that has been placed at dry consistencies with the aid of vibrators, if the surface is protected from excess working due to

keyway construction, the endeavor to embed all coarse aggregate, and the general job traffic until the concrete has hardened.

7. In all cases the new concrete should be preceded by about one-half inch of soft mortar of the same proportions as that in the concrete. When accessible, this should be scrubbed into the surface of the joint with wire brooms. In column forms and deep narrow forms where one-half inch of mortar may seem inadequate, it is preferable to follow the mortar with several inches of concrete containing mortar in excess of that in the usual mix and possibly containing coarse aggregate somewhat reduced in maximum size rather than use large quantities of straight mortar.

Finishing of unformed surfaces

8. For the most durable results in any type of finished, unformed concrete surfaces, the following general requirements should be made. Concrete should be used of the stiffest consistency that can be properly placed and worked or vibrated in accordance with the finishing process adopted. In the initial operations of screeding, floating, and first troweling, the surface of the concrete should be worked *as little as possible* in obtaining the desired result. Each step in the finishing operation from the secondary floating to the final operation should be delayed *as long as possible* and yet permit the desired result to be obtained. The use of any finishing tool in any area where water has accumulated should be prohibited. Dry topping and mortar topping should be avoided. The surface of the concrete should be directly finished to the texture desired.

Cold weather concreting

9. For concrete in sections thinner than would be called mass concrete, when the weather is such that freezing of the concrete may occur, arrangements should be made to heat the mixing water and the aggregate so that the temperature of the fresh concrete in place is approximately 70F. At this temperature setting commences at once and, due to the heat liberated by hydration, will progress at a normal rate, provided the work is protected for several days from dissipation of heat more rapidly than additional heat is generated. This procedure is eminently preferable to that of placing the concrete cold at any temperature above freezing and then endeavoring to heat it in place by salamanders or other means; however, under severe conditions it may be necessary to surround the work with some form of heating protection in addition to protection by insulation. Steam released under a tarpaulin enclosure is an excellent protection since a moist atmosphere favorable to curing as well as protection from freezing is afforded. Corners, edges and surfaces of concrete are particularly vulnerable to

freezing and the need for their adequate protection cannot be too strongly emphasized. In fact they may be frosted when the dry bulb temperature is above 32 F. if the wet bulb temperature reaches 32 F. In connection with cold weather protection, such precautions should be taken that the curing of the concrete will not be impaired and that no portions of the work will become overheated. It is important that there should be a curing period of sufficient length at temperatures above freezing so that when it is exposed to temperatures below freezing at the end of the curing period the concrete will not be injured.

10. With adequate protection of the surfaces from freezing, the minimum temperature of freshly mixed mass concrete may be permitted to be as low as 40F. when placed, because the heat of hydration is lost much more slowly from this type of concrete.

Hot weather concreting

11. For best ultimate quality, concrete should be placed at the lowest practicable temperature during hot weather. Effective results can usually be accomplished by concreting only at night, sprinkling or cooling the aggregate, avoiding hot cement, and using very cold mixing water. Any combination or degree of these practices that may be feasible for the particular work are important and should be required. Curing should preferably be obtained by sprinkling or covering with moist burlap for its additional cooling value. If curing must be done by means of black bituminous sealing compounds, they should be given a coat of whitewash promptly so as not to expose the heat absorbing black surface to the sun.

Forms

12. Provisions as to certain types of forms are necessary because, when they are not followed, concrete containing excessive amounts of sand, cement, or water is usually necessary in order to obtain acceptable workmanship. Horizontally moving slip forms should not be used on slab work on which standard types of paving and finishing machines can be used. Preferably such machines should be equipped with an effective form of vibration or mechanical device for working and compacting the concrete. A continually moving slip form, however, should be used in preference to fixed forms whenever possible on slope paving slabs or steeply curved inverts.

13. Workmanship in placing concrete is largely judged by the appearance of the work on removal of the forms, and appearance in itself is an important quality which must usually be obtained when the concrete is placed. It is therefore necessary in these recommendations to mention other requirements for satisfactory results.

14. With the general adoption of vibration of concrete, forms must not only be built substantially, but they must be tight, since, otherwise, unsightly sand streaks and gravel pockets will be caused by loss of mortar made unusually liquid during vibration. Exceptional care should be exercised to insure that all form panel joints, corners, and connections and all seams between all types of sheathing are completely tight immediately prior to placing the concrete. Because of their superior tightness where they penetrate the forms, tie rods are recommended in preference to tie wires. Where wires must be used holes for them should be as small as possible to keep mortar leakage at a minimum.

15. Forms should be protected from deterioration, weather and shrinkage prior to concreting by proper oiling or by effective wetting. Form surfaces should be clean and of uniform texture. Where they are permissible, re-used forms should be carefully cleaned and oiled. Steel forms should be thoroughly cleaned but never sand blasted nor abraded to bright metal. Where "peeling" is encountered with steel forms, leaving the cleaned, oiled, forms in the sun for a day, or vigorously rubbing the affected areas with liquid paraffin will usually improve the condition. It is generally less expensive and more satisfactory to obtain the desired surface effect by proper treatment and preparation of the forms than it is to obtain it by working over the concrete after the forms are removed.

VI. CLOSURE

1. In general, all of the foregoing features should be considered, insofar as applicable, in connection with each job classification, regardless of type or capacity of equipment.

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ACI Standard

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)*

Reported by ACI Committee 613

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INTRODUCTION

1. General comments

The design of concrete mixes is the determination of the most economical and practicable combination of available aggregates, cement, water, and, in some cases, admixture, that will produce a mixture, having the required degree of workability, and which will develop required qualities on hardening.

The most practical procedure for determining the final proportions of concrete for a given purpose is actual trial and adjustment on the job. The problem thus becomes a matter of selecting a trial mix, for starting concrete operations, that will require the least adjustment on the job.

The design of trial mixes can be accomplished most effectively by laboratory investigation of the concrete-making properties of the particular materials to be used. For work of considerable magnitude, for unusual conditions or materials, or where strength (especially flexural

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strength) is a controlling factor, such laboratory tests are particularly desirable. The minimum laboratory determinations that will permit effective trial mix design are tests of the grading, specific gravity, absorption, and total moisture content of the aggregates. The specific gravity of the cement may be taken as 3.15, without appreciable sacrifice in accuracy. Methods for determining these properties are prescribed in 1 of the appendix.

Important factors which should be determined by tests of concrete mixes in the laboratory include:

(a) Relation between water-cement ratio and strength (compressive or flexural).

(b) Variations in workability characteristics for various combinations of the ingredient materials.

(c) Unit water contents and cement requirements for various aggregate gradings.

In addition to supplying data for more precise design of trial mixes, the laboratory tests provide information that is useful in adjusting concrete mixes in the field and for determining the relative costs of concretes made with different combinations of materials. The recommended procedure for making laboratory mix tests is outlined in 1 of the appendix.

When it is impracticable to conduct laboratory concrete tests, trial mixes for starting concrete operations can be selected by judicious application of known basic concepts and established empirical relationships.

RECOMMENDED PROCEDURE

2. Outline of procedure

The six steps involved in the determination of a trial mix for initial field use are:

(a) Select the water-cement ratio from test data, experience, or established relationships, to meet the specified requirements for durability and strength (compressive or flexural).

(b) Select the limits of slump that will permit proper handling and consolidation of the concrete under the job conditions involved.

(c) Determine the largest size of available aggregate suitable for use under job conditions.

(d) Estimate, from test data, experience, or established relationships, the minimum percentage of sand that will provide the proper degree of workability.

(e) Estimate the amount of water per cubic yard of concrete that will be required to fulfill the conditions of steps (b), (c), and (d).

(f) Compute the trial mix proportions that conform with the factors determined in steps (a) to (e). Make such adjustments in the trial mix, on the job, as may be necessary.

TABLE 1—NET WATER-CEMENT RATIOS FOR VARIOUS TYPES OF CONSTRUCTION AND EXPOSURE CONDITIONS**

Type or Location of Structure	Severe or Moderate Climate, Wide Range of Temperature, Rain, and Long Freezing Spells or Frequent Freezing and Thawing										Mild Climate, Rain or Semiarid Rarely Snow or Frost									
	Thin Sections				Moderate Sections				Heavy and Mass Sections		Thin Sections				Moderate Sections				Heavy and Mass Sections	
	Reinf.		Plain		Reinf.		Plain				Reinf.		Plain		Reinf.		Plain			
	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack
A. At the water line in hydraulic or water-front structures or portions of such structures where complete saturation or intermittent saturation is possible, but not where the structure is continuously submerged:																				
In sea water	0.44	5	0.49	5½	0.49	5½	0.53	6	0.53	6	0.44	5	0.49	5½	0.49	5½	0.53	6	0.53	6
In fresh water	0.49	5½	0.53	6	0.53	6	0.58	6½	0.58	6½	0.49	5½	0.53	6	0.53	6	0.58	6½	0.58	6½
B. Portions of hydraulic or water-front structures some distance from the water line, but subject to frequent wetting:																				
By sea water	0.49	5½	0.53	6	0.53	6	0.53	6	0.53	6	0.49	5½	0.58	6½	0.58	6½	0.62	7	0.62	7
By fresh water	0.53	6	0.58	6½	0.58	6½	0.58	6½	0.58	6½	0.53	6	0.62	7	0.62	7	0.67	7½	0.67	7½
C. Ordinary exposed structures, buildings and portions of bridges not coming under above groups	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7	0.53	6	0.62	7	0.62	7	0.67	7½	0.67	7½
D. Complete continuous submergence:																				
In sea water	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7
In fresh water	0.58	6½	0.62	7	0.62	7	0.67	7½	0.67	7½	0.58	6½	0.62	7	0.62	7	0.67	7½	0.67	7½
E. Concrete deposited through water	*	*	*	*	0.49	5½	0.49	5½	0.49	5½	*	*	*	*	0.49	5½	0.49	5½	0.49	5½
F. Pavement slabs directly on ground:																				
Wearing slabs	0.49	5½	0.53	6	*	*	*	*	*	*	0.53	6	0.58	6½	*	*	*	*	*	*
Base slabs	0.58	6½	0.62	7	*	*	*	*	*	*	0.62	7	0.67	7½	*	*	*	*	*	*

G. Special case: (a) For concrete not exposed to the weather, such as interiors of buildings and portions of structures entirely below ground, no exposure hazard is involved, and the water-cement ratio should be selected on the basis of the strength and workability requirements.

*These sections not practicable for the purpose indicated.
**Adapted from Table 1 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete

3. Selection of water-cement ratio

The water-cement ratio should be chosen on the basis of (a) the required durability, and (b) the strength specified.

The maximum water-cement ratio for the type of construction and exposure conditions involved should be selected from Table 1. The water-cement ratio for the specified strength should be determined by laboratory tests, as described in 1 of the appendix. When it is not practicable to make such tests, the water-cement ratio required for the specified *compressive* strength may be taken directly from Table 2, which gives a series of conservative values derived from a large number of tests with typical materials.

TABLE 2—COMPRESSIVE STRENGTH FOR VARIOUS WATER-CEMENT RATIOS*

Net Water-Cement Ratio		Probable Strength at 28 Days (Pounds per Square Inch)
By Weight	Gallons per Sack of Cement	
0.44	5	5,000
0.49	5½	4,500
0.53	6	4,000
0.58	6½	3,600
0.62	7	3,200
0.67	7½	2,800
0.71	8	2,500
0.75	8½	2,000

*Adapted from Table 2 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete. The strengths listed are based on the use of normal portland cement.

A corresponding table of values for flexural strength is not given because of the wider range in flexural strength which may be obtained with a given water-cement ratio for different materials. Where flexural strength is specified, the required water-cement ratio should be determined by laboratory tests, as previously stated.

The lower of the two water-cement ratios, (a) that required for durability as shown in Table 1, or (b) that required for the specified strength as determined by test or from Table 2, should always be used.

4. Selection of limiting slump and maximum size of aggregate

It is advisable to use the lowest slump compatible with proper placing. Table 3 gives the limiting values recommended for various types of construction.

For economic and other reasons, the maximum size of aggregate should be as large as practicable and available, but should not exceed two-thirds of the minimum clear distance between reinforcement. Recommended limits are given in Table 4.

TABLE 3—RECOMMENDED SLUMPS FOR VARIOUS TYPES OF CONSTRUCTION*

Type of Construction	Slump in Inches**	
	Maximum	Minimum
Reinforced foundation walls and footings	5	2
Plain footings, caissons, and substructure walls	4	1
Slabs, beams, and reinforced walls	6	3
Building columns	6	3
Pavements	3	2
Heavy mass construction	3	1

*Adapted from Table 4 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.
**When high-frequency vibrators are used, the values given should be reduced about one-third.

TABLE 4—MAXIMUM SIZE OF AGGREGATE RECOMMENDED FOR VARIOUS TYPES OF CONSTRUCTION

Minimum Dimension of Section (Inches)	Maximum Size of Aggregate*, in Inches, for:			
	Reinforced Walls, Beams, and Columns	Unreinforced Walls	Heavily Reinforced Slabs	Lightly Reinforced or Unreinforced Slabs
2½ - 5	½ - ¾	¾	¾ - 1	¾ - 1½
6 - 11	¾ - 1½	1½	1½	1½ - 3
12 - 29	1½ - 3	3	1½ - 3	3
30 or more	1½ - 3	6	1½ - 3	3 - 6

*Based on square openings.

5. Estimating the percentage of sand and unit water content

For a given set of materials and water-cement ratio, the unit water content (water required per cubic yard of concrete) is the most important basic factor affecting the quality of concrete. The optimum percentage of sand for a concrete mix is that quantity which will result in the lowest unit water content and also provide the required degree of workability, with an adequate margin to prevent difficulties from variations in working conditions and materials. Obviously, for a given water-cement ratio, the mix requiring the lowest water content will also require the least cement.

The optimum percentage of sand can be determined to best advantage on the job, under working conditions. However, laboratory mix tests with the materials to be used in the work will provide information permitting a close approximation to the proper proportion. The committee recognizes that there are several satisfactory methods for proportioning sand and coarse aggregate for trial concrete mixes. Where conditions

or personal preference dictates, use may be made of any of the basically sound methods described in the current literature (see bibliography).

It is recommended that the proportions for trial concrete mixes be determined on the basis of the estimated unit water requirements. When laboratory tests cannot be made, Table 5 may be employed. The values given in Table 5, which are average values for aggregate gradings within acceptable limits were derived from a composite of data, information, and experience from many sources.

TABLE 5—APPROXIMATE SAND AND WATER CONTENTS PER CUBIC YARD OF CONCRETE

Based on aggregates of average grading and physical characteristics in mixes having a W/C of about 0.57 by weight or 6½ gallons per sack of cement; 3-in. slump, and natural sand having an F.M. of about 2.75.

Maximum Size of Coarse Aggregate	Rounded Coarse Aggregate			Angular Coarse Aggregate		
	Sand Per Cent of Total Aggregate by Absolute Volume	Net Water Content Per Cubic Yard		Sand Per Cent of Total Aggregate by Absolute Volume	Net Water Content Per Cubic Yard	
Inches		Pounds	Gallons		Pounds	Gallons
½	51	335	41	56	360	44
¾	46	310	37	51	335	40
1	41	300	36	46	325	39
1½	37	280	34	42	305	37
2	34	265	32	39	290	35
3	31	250	30	36	275	33
6	26	220	26	31	245	29

Adjustment of Values in Table 5 for Other Conditions

Changes in Conditions Stipulated in Table 5	Effect on Values in Table 5	
	Per Cent Sand*	Unit Water Content*
Each 0.05 increase or decrease in water-cement ratio	±1	0
Each 0.1 increase or decrease in F. M. of sand	±½	0
Each 1 in. increase or decrease in slump	———	±3%
Manufactured sand (sharp and angular)	+3	+15 lb.
For less workable concrete, as in pavements	−3	− 8 lb.

*(+) indicates an increase and (−) a decrease corresponding to the conditions stated in the first column.

6. Computation of trial mix

Assume a project which involves an ordinary heavily reinforced retaining wall having a minimum thickness of 8 in., and also a pavement slab. The concrete will be subjected to severe climatic exposure. The wall and slab have been designed on the basis of compressive and flexural strengths

at 28 days of 4,000 and 600 psi, respectively. A trial concrete mix is required, either to start a series of laboratory mix tests or, if laboratory tests cannot be made, to start concrete construction in the field. The concrete in the retaining wall will be consolidated by vibration, and the following information regarding the materials to be used in the work is available:

Cement—Type I, average characteristics.
Specific gravity—3.15 (assumed).

Sand—River sand, damp.
Medium fineness and grading.
Specific gravity—2.65 (saturated, surface dry).
Free moisture content—5 per cent by weight.

Coarse aggregate—Crushed stone (saturated surface-dry.)
Reasonably well graded, angular.
Specific gravity—2.55 (saturated, surface dry.)

From Table 1 it is found that a water-cement ratio of 0.53 is the maximum which should be used for the type of structures and service conditions involved. Table 2 indicates that a water-cement ratio of 0.53 will provide the required compressive strength of 4,000 psi. The water-cement ratio required for the specified flexural strength of 600 psi can be established only by laboratory tests such as outlined in 1-c of the appendix. From Tables 3 and 4 it is concluded that for the concrete in the retaining wall a three-in. slump and 1½-in. maximum aggregate will be satisfactory.

On the basis of information given in Table 5 it is estimated that the percentage of sand for the trial mix should be 42 per cent, by absolute volume of the total aggregate, and that 305 pounds (about 37 gallons) of water will be required per cubic yard of concrete. (Actually the water content may be more or less than the estimated amount as determined by the amount of water required for a three-in. slump when a trial batch is mixed in the laboratory or on the job.)

$$\text{Cement content} = \frac{\text{Net water content}}{\text{water-cement ratio}}$$

$$= \frac{305}{0.53} = 575 \text{ lb. per cu. yd.}$$

$$= \frac{575}{94} = 6.12 \text{ sacks per cu. yd.}$$

$$\left(\begin{matrix}\text{Absolute volume,}\\ \text{water + cement}\end{matrix}\right) = \frac{\text{Water content}}{62.4} + \frac{\text{Cement content}}{\text{specific gravity} \times 62.4} =$$

$$\frac{305}{62.4} + \frac{575}{3.15 \times 62.4} = 7.81 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\begin{pmatrix}\text{Absolute volume,}\\ \text{total aggregate}\end{pmatrix} = 27 - \begin{pmatrix}\text{absolute volume,}\\ \text{water + cement}\end{pmatrix}^{*}$$
$$= 27 - 7.81 = 19.19 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\begin{pmatrix}\text{Absolute volume,}\\ \text{sand}\end{pmatrix} = \text{percent sand} \times \begin{pmatrix}\text{absolute volume,}\\ \text{total aggregate}\end{pmatrix}$$
$$= 0.42 \times 19.19 = 8.06 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\begin{pmatrix}\text{Absolute volume,}\\ \text{coarse aggregate}\end{pmatrix} = \begin{pmatrix}\text{absolute volume,}\\ \text{total aggregate}\end{pmatrix} - \begin{pmatrix}\text{absolute volume,}\\ \text{sand}\end{pmatrix}$$
$$= 19.19 - 8.06 = 11.13 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\begin{pmatrix}\text{sand}\\ \text{content}\end{pmatrix} = \text{absolute volume} \times \text{specific gravity} \times 62.4$$
$$= 8.06 \times 2.65 \times 62.4 = 1{,}333 \text{ lb. per cu. yd. of concrete.}$$

$$\begin{pmatrix}\text{Coarse aggregate}\\ \text{content}\end{pmatrix} = 11.13 \times 2.55 \times 62.4 = 1{,}771 \text{ lb. per cu. yd. of concrete.}$$

$$\text{Trial-mix proportions} = \frac{575}{575} : \frac{1{,}333}{575} : \frac{1{,}771}{575}$$
$$= 1 : 2.32 : 3.08, \text{ say } 1:2.3:3.1$$

This mix provides a starting point for a series of laboratory tests, such as described in 1-c of the appendix, or for field operations in constructing the retaining wall.

The equipment available on the job includes a three-sack batch mixer equipped with a water tank calibrated in gallons and also a water meter graduated in pounds, and batching scales for weighing the separated sizes of sand and coarse aggregate.

Cement is to be batched on the basis of integral sacks and the batch will contain three sacks, or 3 x 94 = 282 lb. of cement, which is a fixed quantity. The batch weights of sand and coarse aggregate for the trial mix will be:

Sand = $1{,}333 \times \frac{282}{575} = 654$ lb. (net), to which must be added the weight of the free moisture in the sand: $654 + (0.05 \times 654) = 687$ lb. damp sand.

Coarse aggregate = $1{,}771 \times \frac{282}{575} = 869$ lb.

7. Adjustment of trial mix

A few batches of concrete are mixed on the job, and the average quantity of water added per batch for the desired three-in. slump is 13.1 gal. or 13.1 x 8.34 = 109 lb.

The free water in the sand is: $0.05 \times 654 = 33$ lb.
The net water in the mix is: $109 + 33 = 142$ lb.

The net water-cement ratio is: $\frac{142}{282} = 0.50$ by weight.

*Air voids assumed to be negligible. When the air content is appreciable, as when air entraining agents are used, a suitable allowance should be made, considering the air as replacing an equal volume of sand.

Since a water-cement ratio of 0.53 is desired (water content of $0.53 \times 282 = 150$ lb. for a three-sack batch), the batch requires less water than was estimated.

The absolute volume of the field batch = volume of water + absolute volumes of cement + sand + coarse aggregate =

$$\frac{142}{62.4}+\frac{282}{3.15 \times 62.4}+\frac{654}{2.65 \times 62.4}+\frac{869}{2.55 \times 62.4} = 13.13 \text{ cu. ft.}$$

The correct unit water content is: $142 \times \frac{27}{13.13} = 292$ lb. per cu. yd.

Using the corrected value for unit water content, as determined by the trial batches, the batch quantities for the adjusted mix having a water-cement ratio of 0.53, may be computed in the same manner as for the trial mix. This procedure should result in concrete having the desired three-in. slump because it is a recognized fact that for a given slump and aggregate grading, substantially the same unit water content is required for any water-cement ratio.*

The adjusted mix for 292 lb. of water per cubic yard and a 0.53 water-cement ratio is 1:2.46:3.27; say 1:2.5:3.3.

After the initial adjustment for water-cement ratio has been made and after stabilized operating conditions have been established, the optimum percentage of sand should be determined. This may be done by trial, noting the water requirement for each percentage of sand tried, until the lowest percentage compatible with proper workability, with a safe working margin, is established. The mix is again adjusted to maintain the correct water-cement ratio.

Rich mixes normally require less sand than lean mixes. After the optimum percentage of sand has been determined for a given mix, cement and sand may be interchanged by absolute volumes (keeping the sum of their absolute volumes the same) without affecting the water content or slump. For practical purposes a change in cement content of one sack per cubic yard of concrete will permit a change in sand percentage of about two.

Changes in consistency may be made by simply increasing or decreasing the amount of water to provide the desired slump, with appropriate adjustment to maintain the specified water-cement ratio.

The job mix should not be adjusted for minor fluctuations in water-cement ratio. A variation of ± 0.02 (by weight) to maintain a constant slump is considered normal.

*A short-cut method for adjusting a mix for change in water-cement ratio is illustrated in 3 of the appendix.

APPENDIX

1. Laboratory tests

As stated in the introduction, the design of trial mixes can be accomplished most effectively by laboratory investigations: *First*, to determine the basic physical properties of the materials to be used; and *second*, to establish certain fundamental relationships, including: (1) Relation between water-cement ratio and strength; (2) variations in workability characteristics for various combinations of the ingredient materials; and (3) unit water contents and cement requirements for various aggregate gradings.

The extent of the investigations desirable for any given job will depend on the size, importance, and service conditions of the structures involved. Details of the laboratory program will also vary depending on the facilities available and individual preferences.

(a) *Physical properties of cement*—The fineness and chemical composition of cement influence the workability and strength development of concrete; however, the only property of cement directly required in computations for the design of concrete mixes and batch quantities is specific gravity. The specific gravity of cement may be assumed to be 3.15 without introducing appreciable error in the mix computations, or it may be determined by test. A sack of cement weighs 94 pounds. It is, of course, desirable to assemble complete information concerning the cement to be used, including both chemical and physical properties.

The sample of cement should be obtained from the mill which will supply the job or preferably from the job itself, if time permits. The sample should contain ample quantity for the tests contemplated with a liberal margin of excess for additional tests that might be desirable. The test cement should be shipped and stored in air-tight containers or at least in moisture-proof packages.

(b) *Physical properties of aggregate*—Specific gravity, absorption, and moisture content are the essential physical properties of aggregates required in mix design computations. The weight per cubic foot and void content of the aggregate will be useful in analyzing concrete mixes.

For large or special types of work, various additional tests of aggregate materials, including petrographic analyses, chemical reactivity tests, soundness tests, abrasion tests, and tests for various deleterious substances, may be justified in connection with mix design studies, because all such tests yield information of value in judging the ultimate quality of the concrete and in selecting appropriate mix design factors.

Aggregate gradation, or particle size distribution, is a major factor controlling the unit water requirement, percentage of sand and cement content of concrete mixes for a given degree of workability. Some stand-

ard for comparing different aggregate gradings in concrete mix tests is needed to properly evaluate such factors as workability, cement and water requirements, benefits to be derived from processing or the use of admixtures, and the relative economy of alternative aggregate sources. Numerous "ideal" aggregate grading curves have been proposed but a universally accepted standard has not been developed. Experience and individual conditions and judgment must continue to play important roles in determining "acceptable aggregate gradings."

Variations in sand grading may be accomplished by (1) separation of the sand into two or more size fractions and recombination in suitable proportions, (2) adding or removing materials to balance the grading, or (3) reducing excess coarse material by grinding. Variations in coarse aggregate grading may be accomplished by (1) crushing excess material from coarser fractions or oversize material, (2) wasting excess materials from other fractions, or (3) a combination of these methods. Insofar as grading limitations and consideration of cement economy permit, the relative proportions of the various sizes of coarse aggregate used in the trial mixes should be governed by the natural grading of the materials. Whatever processing is done in the laboratory for the purpose of comparing mixes should be practical from the standpoint of economy and job operation.

Samples of aggregates for concrete mix tests should be representative of the aggregate selected for use in the work. The coarse aggregate should be cleanly separated into the required size fractions to provide uniform control of mix proportions.

(c) *Concrete mix tests*—The example used in section 6 of the recommended procedure requires flexural and compressive strengths at 28 days of 600 and 4,000 psi, respectively. A minimum series of concrete mix tests to establish the relationships needed for the selection of appropriate mix design factors is illustrated in Tables 6 and 7.

The trial mix, as calculated in section 6, provides a starting point for the laboratory test series. Five additional mixes are then made with the water-cement ratio varying over a total range of 0.20 to establish water-cement-ratio-strength relationships. For this set of mixes the percentage of sand is varied according to the water-cement ratio, or richness of mix, as indicated in the recommended procedure.

The strength data obtained from the concrete mix tests are given in Table 7. With such information, for the specific materials to be used on the job, it is unnecessary to use the empirical values and relationships for average conditions as given in Tables 2 and 5 of the recommended procedure. The test data show that a water-cement ratio of

TABLE 6—TYPICAL MINIMUM PROGRAM OF CONCRETE MIX TESTS

Maximum size aggregate—1½ in. Required slump approximately 3 in.

Mix No.	Actual Net W/C by Wt.	Sand % of Total Aggregate (by Abs. Volume)	Mix by Weight Based on Saturated Surface-Dry Aggregates	Unit Weight Lbs. per Cu. Ft.	Slump, In.	Unit Water Content Lbs. per Cu. Yd.	Cement Content Lbs. per Cu. Yd.	Workability			
								Rod-ability	Finish	Segre-gation	Remarks
1	.50	42	1:2.32:3.08	148.2	3	292	584	Good	Very good	None	Very workable, could use less sand.
2	.43	38	1:1.79:2.81	149.0	2¾	287	667	Good	Excellent	None	Excellent mix, slightly sticky,
3	.48	39	1:2.09:3.15	148.6	3¼	287	598	Good	Excellent	None	Excellent mix
4	.53	40	1:2.40:3.47	148.2	3	287	541	Good	Very good	None	Excellent mix.
5	.58	41	1:2.72:3.77	148.0	2¾	287	495	Good	Good	None	Good mix.
6	.63	42	1:3.06:4.07	147.9	3¼	287	456	Good	Fair	None	Good mix, lacks cohesiveness

0.50 is required to provide the specified flexural strength of 600 psi. A maximum water-cement ratio of 0.53, required for durability, will result in workable concrete that will develop more than the 4,000 psi compressive strength specified for the retaining wall. The trial field mixes may now be calculated as before or taken directly from Tables 6 and 7 by interpolation.

TABLE 7—STRENGTH DATA FROM CONCRETE MIX TESTS SHOWN IN TABLE 6

Average Unit Strengths at 28 Days

Mix No.	Compression psi of 3 6x12-in. Cylinders	Flexural psi of 5 6x6x21-in. Beams	Compression psi of 2 6x6x6-in. Modified Cubes
1	4,390	610	5,000
2	5,060	680	5,730
3	4,630	625	5,260
4	4,080	570	4,800
5	3,800	525	4,390
6	3,440	490	3,950

In conducting laboratory mix tests, it will seldom be found, even with experienced operators, that the desired conditions are obtained so precisely for every mix as indicated in Table 6. Some trial and error and rerun of mixes may be anticipated. Furthermore, it should not be expected that field materials, conditions, and equipment will permit an exact check of laboratory results, and adjustment of the selected trial mix on the job will probably be desirable. Closer agreement between laboratory and field results will be assured if machine mixing is employed in the laboratory, and this procedure is recommended. If laboratory mixer equipment is not available, hand mixing can be used. Similarly, any processing of materials in the laboratory should simulate as closely as practicable the corresponding treatment in the field.

The minimum series of tests illustrated in Tables 6 and 7 may be expanded as the size, cost, and special requirements of the work might warrant. Alternative aggregate sources, different sand and coarse aggregate gradings, different percentages of sand, different types or brands of cement, admixtures, mixes for various maximum sizes of aggregate, and concrete performance in durability, volume change, and thermal property tests, are some of the variable factors that may require consideration in more extensive programs.

(d) *Test methods*—In conducting laboratory tests, it is recommended that the latest issue of the following listed standards be used.

Cement

Sampling and physical testing of portland cement—ASTM Designation: C77.
Chemical analysis of portland cement—ASTM Designation: C114.
Fineness of portland cement by means of the turbidimeter—ASTM Designation: C115
Compressive strength of portland cement mortars—ASTM Designation: C109.
Autoclave expansion of portland cement—ASTM Designation: C151.

Water

Quality of water to be used in concrete—AASHO Designation: T26.

Aggregate

Abrasion of coarse aggregate—Los Angeles machine—ASTM Designation: C131.
Abrasion of rock by use of the Deval machine—ASTM Designation: D2.
Abrasion of stone and slag by use of the Deval machine—AASHO Designation: T3.
Abrasion of gravel by use of the Deval machine—ASTM Designation: D289.
Toughness of rock—ASTM Designation: D3.
Clay lumps in aggregates—ASTM Designation: C142.
Coal and lignite in sand—ASTM Designation: C123.
Percentage of shale in aggregate—AASHO Designation: T10
Organic impurities in sand—ASTM Designation: C40.
Material finer than No. 200 sieve—ASTM Designation: C117.
Sieve analysis of fine and coarse aggregate—ASTM Designation: C136.
Soundness—Sodium and magnesium sulfate method—ASTM Designation: C88.
Soundness—Freezing and thawing method—ASTM Designation: C137.
Specific gravity and absorption—Coarse aggregate—ASTM Designation: C127.
Specific gravity and absorption—Fine aggregate—ASTM Designation: C128.
Surface moisture in fine aggregate—ASTM Designation: C70.
Unit weight of aggregate—ASTM Designation: C29.
Voids in aggregate for concrete—ASTM Designation: C30.
Structural strength of fine aggregate using constant water-cement ratio mortar—ASTM Designation: C87.

Concrete

Machine mixing of concrete in laboratory—Bureau of Reclamation Concrete Manual—Designation: 28.
Sampling of fresh concrete—ASTM Designation: C172.
Consistency by slump—ASTM Designation: C143.
Consistency by flow table—ASTM Designation: C124.
Yield of concrete (includes unit weight test)—ASTM Designation: C138.
Air content of freshly mixed concrete—ASTM Designation: C173.
Making and storing compression test specimens in the field—ASTM Designation: C31.
Compressive strength—ASTM Designation: C39.
Compressive strength—Modified cube method—ASTM Designation: C116.
Flexural strength—ASTM Designation: C78.
Volume change—ASTM Designation: C157.
Absorption of concrete—AASHO Designation: T25.
Securing and testing specimens of hardened concrete from the structure—ASTM Designation: C42.
Cement content of hardened concrete—ASTM Designation: C85.

2. Other methods for computation of trial mixes

Many charts, rules, and tables have been devised for use in concrete mix computations. These devices, which are largely timesavers, range in accuracy from that of the method described in the recommended procedure to rough tables from which a mix may be chosen that may be safely used, in small work, without adjustment.

Nomographic chart—The nomograph illustrated in Fig. 1 is based on the same principles as the recommended procedure, being merely a graphic solution of these fundamentals. An explanation of the use of the chart follows—the illustration is the same as the one employed in section 6:

Water-cement ratio = 0.53 by weight.
Unit water content = 305 lb. per cu. yd.
Proportion of sand = 42 per cent.
Specific gravity: Sand = 2.65, coarse aggregate = 2.55.

Place a transparent straightedge across the nomograph—see line (1) on chart—to connect scale points for water-cement ratio of 0.53 and water content of 305.

Read off chart: Parts of aggregate by weight = 5.5 (for 2.65 sp. gr.)
Cement content = 1.53 bbl. per cu. yd. (6.12 sacks per cu. yd.)

Parts of sand by weight = 42 per cent of 5.5 = 2.3.

Parts of coarse aggregate by weight = 5.5 — 2.3 = 3.2 (for 2.65 sp. gr.) or $\frac{2.55}{2.65} \times 3.2 = 3.1$ (for 2.55 sp. gr.)

Trial mix = 1:2.3:3.1 by weight.

3. Other methods for adjustment of trial mixes

Short-cut methods for adjusting concrete mixes can be used with sufficient accuracy for most practical purposes.

(a) *The nomographic chart*—The nomographic chart illustrated in Fig. 1 can be used, either as shown or in any preferred variation. In the example which follows, the mix and required adjustment are the same as those employed in the recommended procedure.

Connect scale points on nomograph—line (2) on chart—for water-cement ratio of 0.50 and parts of aggregate of 5.5 (2.65 sp. gr.).

Read off chart: Water content = 292 lb. per cu. yd.

Connect scale points on nomograph—line (3) on chart—for water content of 292 and water-cement ratio of 0.53.

Read off chart: Parts of aggregate = 5.9 (for 2.65 sp. gr.)
Cement content = 1.46 bbl. per cu. yd. (5.84 sacks per cu. yd.)

Parts of sand = 42 per cent of 5.9 = 2.5.

Parts of coarse aggregate = 5.9 — 2.5 = 3.4 (for 2.65 sp. gr.) = $\frac{2.55}{2.65} \times 3.4 = 3.3$ (for 2.55 sp. gr.)

Adjusted mix: 1:2.5:3.3 by weight.

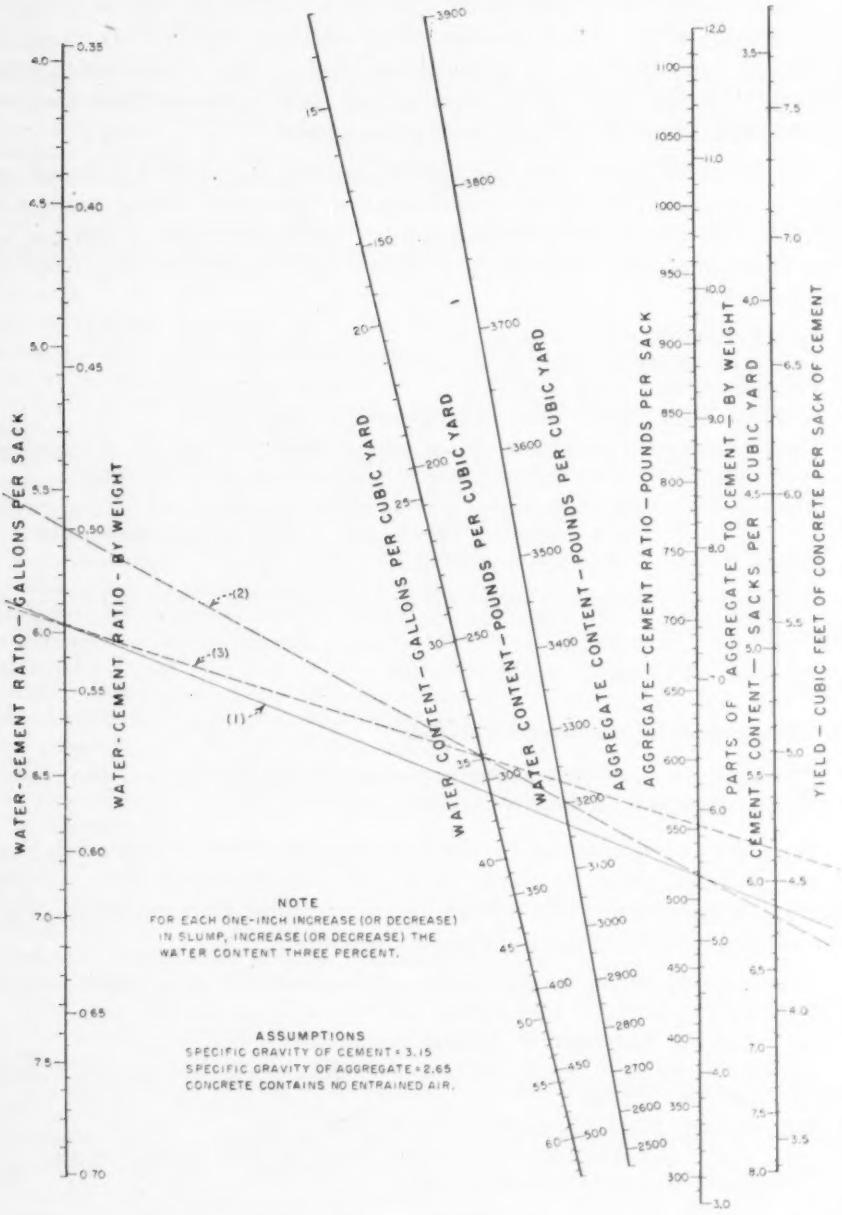
WATER-CEMENT RATIO — GALLONS PER SACK
WATER-CEMENT RATIO - BY WEIGHT
WATER CONTENT — GALLONS PER CUBIC YARD
WATER CONTENT — POUNDS PER CUBIC YARD
AGGREGATE CONTENT — POUNDS PER CUBIC YARD
AGGREGATE — CEMENT RATIO — POUNDS PER SACK
PARTS OF AGGREGATE TO CEMENT — BY WEIGHT
CEMENT CONTENT — SACKS PER CUBIC YARD
YIELD — CUBIC FEET OF CONCRETE PER SACK OF CEMENT
(1)
(2)
(3)
NOTE
FOR EACH ONE-INCH INCREASE (OR DECREASE) IN SLUMP, INCREASE (OR DECREASE) THE WATER CONTENT THREE PERCENT.
ASSUMPTIONS
SPECIFIC GRAVITY OF CEMENT = 3.15
SPECIFIC GRAVITY OF AGGREGATE = 2.65
CONCRETE CONTAINS NO ENTRAINED AIR.

Fig. 1—Chart for design and adjustment of concrete mixes

(b) *Rules for adjustment of mixes*—For those who prefer a procedure for adjusting mixes that is not dependent on charts, absolute volume computations or detail tabulations, the rules in Table 8 may be found useful.

TABLE 8—RULES AND INCREMENTS FOR MIX ADJUSTMENTS
(Constant Aggregate Grading and Specific Gravity)

To Change	To Hold	Corresponding Adjustments				
		Slump	W/C By Weight	Unit Water Content	Total (1) Mix Parts By Weight	Cement (2) Content Bbl. per Cu. Yd.
CASE I						
W/C	Slump	Constant	Variable	Constant	*Directly proportional to W/C	Inversely proportional to W/C
This adjustment amounts to replacing aggregate with cement (or vice versa) in a unit volume of concrete						
CASE II						
Slump	W/C	1-in. increase 1-in. decrease	Constant Constant	+3% −3%	−0.25 +0.25	+0.04 −0.04
This adjustment amounts to replacing aggregate with cement-and-water paste (or vice versa) in a unit volume of concrete						
CASE III						
W/C	Mix Parts	1-in. increase 1-in. decrease	+0.02 −0.02	+3% −3%	Constant Constant	−0.01 +0.01
This adjustment amounts to replacing cement and aggregate with water (or vice versa) in a unit volume of concrete						
CASE IV						
% of sand by Weight	Slump and W/C	Constant	Constant	For each change of 1 in % of sand change water content 2.5 lbs. in same direction	Variable (4)	For each change of 1 in % of sand change cement content 0.01 bbl. in same direction

*For all practical purposes.

(1) Total mix parts equals total parts of aggregate and cement.

(2) Approximate cement content for unit weights shown in footnote (3.)

$$\text{Bbl. per cu. yd.} = \frac{\text{(wt. of concrete per cu. ft.) } 27^{(3)}}{\text{(Total mix parts + w/c) } 376}$$

(3) The unit weight of concrete containing aggregate of 2.65 average specific gravity is approximately 148 lb. per cu. ft. for ¾-in. max. aggregate; 150 lb. for 1½-in. max; 152 lb. for 3-in. max.

If the specific gravity of the combined aggregate differs materially from the average value of 2.65, the computed cement content may be corrected by adding or subtracting 0.02 for each 0.05 increase or decrease, respectively in specific gravity.

(4) For each change of 3 in the percentage of sand, change the parts of sand 0.1, and the total parts of aggregate, in the opposite direction, 0.15, 0.20, 0.25, and 0.30 for ¾, 1½, 3, and 6-in. maximum aggregate sizes, respectively.

Application of these rules to the mix adjustment illustrated in section 7, is as follows:

Mix 1:2.3:3.1 by weight; W/C = 0.50.

To adjust the W/C to 0.53, without changing the slump or percentage of sand by weight, Case I, Table 8, would apply.

The new total mix parts $= 6.4 \times \frac{0.53}{0.50} = 6.8$, or 1 part cement to 5.8 parts of aggregate.

Parts of sand $= 5.8 \times \frac{2.3}{2.3 + 3.1} = 2.5$

Parts of coarse aggregate $= 5.8 - 2.5 = 3.3$.

Adjusted mix = 1:2.5:3.3.

Other practical uses for the relationships given in Table 8 will be readily apparent to the concrete technician or engineer.

4. Mixes for small jobs

For very small jobs, involving only a few batches of concrete, where time or personnel are not available to determine mixes in accordance with

TABLE 9—MIXES FOR SMALL JOBS

Maximum Size of Aggregate	Mix Designation	Approx. Sacks Cement per Cu. Yd. of Concrete	Pounds of Aggregate per 1-Sack Batch	
			Sand*	Coarse Aggregate
½-in.	A	7.0	245	170
	B	6.9	235	190
	C	6.8	235	205
¾-in.	A	6.6	235	225
	B	6.4	235	245
	C	6.3	225	265
1-in.	A	6.4	235	245
	B	6.2	225	275
	C	6.1	215	290
1½-in.	A	6.0	235	290
	B	5.8	225	320
	C	5.7	215	345
2-in.	A	5.7	235	330
	B	5.6	225	360
	C	5.4	215	380

*Weights are for dry sand. If sand is damp, increase weight of sand 10 pounds for one-sack batch, and if very wet, add 20 pounds for one-sack batch. For slag, use 85 per cent of coarse aggregate weights shown.

These mixes do not apply for lightweight aggregate.

Procedure: Select the proper maximum size of aggregate and then, using mix B, add just enough water to produce a sufficiently workable consistency. If the concrete appears to be undersanded use mix A, and if it appears to be oversanded use mix C.

the recommended procedure, the mixes in Table 9 will provide concrete that is amply strong, dense, and durable if the amount of water added at the mixer is never so large as to make the concrete sloppy. These mixes have been predetermined in conformity with the recommended procedure by assuming conditions applicable for the average small job using aggregate of average specific gravity.

Three mixes are given for each maximum size of coarse aggregate. (Table 4 may be used as a guide in selecting an appropriate maximum size of aggregate). Mix B, for each size of coarse aggregate is intended for use as a starting mix. If this mix happens to be oversanded, change to mix C; if it is undersanded, change to mix A.

It should be noted that the mixes listed in the table apply where the sand is dry, or appears to be dry. If the sand is moist, or very wet, make the corrections in batch weights described in the note at the foot of the table.

The approximate cement contents in sacks per cubic yard of concrete will be helpful in estimating cement requirements for the job. These are based on concrete that has just enough water in it to permit ready working in the forms without objectionable separation. The concrete should slide off a shovel—not run off.

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ACI Standard

Specifications for Concrete Pavements and Bases (ACI 617-44)*

Reported by ACI Committee 617

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SCOPE

1. Concrete pavement and base

These specifications apply to the construction of portland cement concrete pavement and base under normal conditions, including the preparation of the subgrade, and shall govern unless modified by special provisions to take into account unusual conditions of traffic, subgrade, drainage, exposure and other factors.

MATERIALS

2. Approval of sources of material supply

Portland cement, aggregates and water shall be furnished only from sources of supply approved by the engineer before shipments are started. The basis of approval of such sources shall be the ability to produce materials of the quality and in the quantity required.

3. Portland cement

(*a*) Cement shall conform to the requirements of the current American Society for Testing Materials Standard Specifications for Portland Cement (Designation C 150). Type I cement shall be supplied unless Type II (Moderate Heat of Hydration) or Type III (High Early Strength) is required by the special provisions.

(*b*) Cement shall be protected from the weather and against loss in handling or in transit. Either package or bulk cement may be used.

*Adopted as a Standard of the American Concrete Institute at its 40th Annual Convention, March 2, 1944 as reported by Committee 613; ratified by Letter Ballot July 28, 1944. (With editorial revisions from report as published ACI JOURNAL, Nov. 1943).

4. Aggregates

(*a*) Aggregates shall conform to the requirements of the current ASTM Standard Specifications for Concrete Aggregates (Designation C 33).

Note: It will be necessary to include limits in these specifications to meet local conditions.

Coarse aggregate shall be furnished in two separate sizes with the separation at the ¾-in. sieve when combined material graded from No. 4 to 1½ in. is required, and at the 1-in. sieve when combined material graded from No. 4 to 2 in. is required.

(*b*) In arranging for storage of aggregates, positive means shall be used to prevent the inclusion of foreign material. Aggregates shall not be placed upon the finished subgrade. Aggregates of different kinds and sizes shall be placed in different stock piles. Stock piles of coarse aggregates shall be built up in successive horizontal layers not more than 3 ft. thick. Each layer shall be completed before the next is started. Should segregation occur, the aggregates shall be remixed to conform to the grading requirements.

(*c*) Frozen aggregates or aggregates containing frozen lumps shall be thawed before use. Washed aggregates and aggregates produced or manipulated by hydraulic methods shall be allowed to drain for at least twelve hours before use. Stock piles or cars and barges equipped with weep holes are considered to offer suitable opportunity for drainiage.

5. Water

Water used in mixing or curing concrete shall be clean and free from injurious amounts of oil, salt, acid, vegetable or other substances harmful to the finished product. Sources of water shall be maintained at such a depth, and the water shall be withdrawn in such a manner (by enclosing pump intake, etc.), as to exclude silt, mud, grass or other foreign materials.

6. Reinforcement steel and accessories

(*a*) *Steel wire fabric.* Steel wire fabric reinforcement shall conform to the requirements of the current ASTM Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement (Designation: A 185).

(*b*) *Bar mats.* The steel in bar mats shall conform to the requirement of the current ASTM Standard Specifications for Bar or Rod Mats for Concrete Reinforcement (Designation: A 184). Members shall be of the size and spacing shown on the plans. All intersections of longitudinal and transverse bars shall be securely wired, clipped or welded together.

(*c*) *Dimensions of wire fabric and bar mats.* The width of fabric sheets or bar mats shall be such that, when properly placed in the work, the extreme longitudinal members of the sheet or mat will be located not less

than 3 in. nor more than 6 in. from the edges of the slab. The length of fabric sheets or bar mats shall be such that, when properly placed in the work, the reinforcement will clear all transverse joints by not less than 2 in. nor more than 4 in. as measured from the center of the joint to the tip ends of the longitudinal members of the sheet or mat.

(*d*) *Bars.* Reinforcement bars shall conform to the requirements of the current ASTM Standard Specifications for Billet-Steel Bars for Concrete Reinforcement (Designation: A 15) or for Rail-Steel Bars for Concrete Reinforcement (Designation: A 16). Bars depending upon bond for their effectiveness shall be free from excessive rust, scale, or other substances which prevent the bonding of the concrete to the reinforcement.

(*e*) *Tie bars.* Tie bars shall be deformed steel bars conforming to the requirements of the specifications for reinforcement bars except that rail steel bars shall not be used where they are to be bent and restraightened.

(*f*) *Dowels and sleeves.* Dowels shall be plain round bars conforming to the requirements of the specifications for reinforcement bars.

When metal sleeves are used they shall cover the ends of the dowels for not less than 2 in. nor more than 3 in. The sleeve shall be closed at one end. It shall have a suitable stop to hold the end of the bar at least 1 in. from the closed end of the sleeve. It shall be of such rigid design that the closed end will not collapse during construction.

(*g*) *Chairs.* Chairs for holding tie rods in correct position while the concrete is being placed shall be of metal, slightly rounded and tapered at one end. The minimum dimensions shall be: Thickness 16 gauge (U. S. Standard Gauge 1893), length 12 in., width $1\frac{3}{4}$ in. measured along the metal.

(*h*) *Stakes.* Stakes used to support expansion joint fillers shall be channel- or U-shaped, at least $\frac{3}{4}$ in. wide and $\frac{3}{8}$ in. deep and of metal not thinner than 16 gauge (U. S. Standard Gauge 1893). They shall be 15 in. long or longer if required to provide proper bearing support.

7. Subgrade paper

Subgrade paper shall conform to the requirements of the current American Association of State Highway Officials Standard Specification for Subgrade Paper, M 74.

8. Expansion joint filler

Expansion Joint Filler shall conform to the requirements of the current ASTM or AASHO Standard Specifications for Preformed Expansion Joint Fillers for Concrete.

9. Metal joint plate and pins

Plates for tongue and groove joints shall be steel or iron not thinner than 16 gauge, (U. S. Standard Gauge, 1893). Each section shall be a

continuous strip of metal not more than 15 ft. long having a width ½ in. less than the depth of the pavement; it shall be provided with an end connection which will hold the ends of strips firmly together. Each section shall be of the specified cross section, and shall be punched for dowels or tie bars, and pins, as shown on the plans.

Pins shall be channel shaped, pressed out of sheet steel of not less than 12 gauge (U. S. Standard Gauge, 1893) and not less than 15 in. long.

10. Cover materials for curing

(*a*) *Burlap.* Burlap shall be made from jute or hemp and, at time of using shall be in good condition, free from holes, dirt, clay, or any other substance which interferes with its absorptive quality. It shall not contain any substance which would have a deleterious effect on the concrete. Burlap shall be of such quality that it will absorb water readily when dipped or sprayed and shall weigh not less than 7 oz. per square yard when clean and dry. Burlap made into mats may be used if care in handling is exercised to avoid marring the finished surface of the concrete.

(*b*) *Cotton mats.* Cotton mats for curing concrete shall conform to the requirements of the current AASHO Standard Specifications for Cotton Mats for Curing Concrete Pavements. (Designation M 73).

11. Testing materials

Materials shall be tested in accordance with methods referred to in the appropriate specifications, except as otherwise specified.

12. Flexural strength tests of concrete as basis of design

Flexural strength tests to be used as the basis for the design of concrete mixtures shall be carried out in accordance with the current ASTM Standard Method of Test for Flexural Strength of Concrete. (Laboratory Method Using Simple Beam with Third Point Loading, Designation C 78).

13. Flexural strength tests of concrete for field control

(*a*) Flexural tests of concrete specimens molded and cured in the field shall be made, whenever feasible, using the testing machine described in ASTM Method C 78. Field Cured Specimens should not be used as a basis for design of mixtures. If another type of testing machine is used, results obtained with it should be correlated with those obtained from the standard apparatus. Apparatus for making flexure tests of concrete shall be so designed as to incorporate the following principles:

(1) The distance between supports and points of load application shall remain constant for a given apparatus.

(2) The load shall be applied normal to the loaded surface of the beam and in such a manner as to avoid eccentricity of loading.

(3) The direction of the reactions shall, at all times during a test, be parallel to the direction of the applied load.

(4) The load shall be applied at a uniform rate and in such a manner as to avoid shock.

(5) The ratio of distance between point of load application and nearest reaction to the depth of the beam shall not be less than one.

Note: The directions of loads and reactions may be maintained parallel by judicious use of linkages, rocker bearings and flexure plates. Eccentricity of loading can be avoided by use of spherical bearings.

(*b*) *Molding, curing and marking.* Concrete for the field test specimens shall be secured in accordance with the current ASTM Method of Sampling Fresh Concrete (Designation: C 172) from the concrete deposited on the subgrade. The specimens shall be molded and finished as described in the current ASTM Method of Test for Flexural Strength of Concrete (Designation C 78). They shall be cured, as nearly as practicable, in the same manner as the pavement concrete. They shall be properly identified as to date of molding and location of pavement represented; weather conditions prevailing at the time of molding shall be noted.

(*c*) *Number of specimens.* At least two beams shall be made for each one thousand square yards of pavement.

14. Specific gravity and absorption of aggregates

(*a*) *Fine aggregate.* The bulk specific gravity and absorption of fine aggregate shall be determined in accordance with the current ASTM Standard Method of Test for Specific Gravity and Absorption of Fine Aggregates. (Designation C 128).

(*b*) *Coarse Aggregate.* The bulk specific gravity and absorption of coarse aggregate shall be determined in accordance with the current ASTM Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate. (Designation C 127).

15. Consistency

Consistency shall be determined in accordance with the current ASTM Standard Slump Test for consistency of concrete. (Designation C 143).

PROPORTIONS OF MATERIALS

16. Basis of proportions

The proportions of water, cement and aggregates shall be in accordance with *Proportions based on design for minimum strength* (Section 17) or in accordance with *Proportions based on uniform cement factor* (Section 18) as specified in the special provisions.

17. Proportions based on design for minimum strength

(*a*) The proportions of cement, fine aggregate, coarse aggregate and water to be used in the mix shall be determined by the Engineer, within the limits of Sections 17(b) and 17(c), by means of laboratory tests of the flexural strength of concrete made with aggregates from the same

sources and of the same gradings as will be employed in the work and that portland cement which is found to produce the lowest strength concrete of any acceptable cement available for the work.

(*b*) The mixture determined upon shall produce workable concrete having a slump of 2 in. to 3 in. for unvibrated concrete, or ½ in. to 1½ in. for vibrated concrete, and a modulus of rupture at 14 days of not less than 550 psi as determined from specimens made, cured and tested in accordance with the current ASTM Method C 78.

(*c*) The resulting proportions shall be such that the mixing water, including free surface moisture on the aggregates but exclusive of moisture absorbed by the aggregates, shall not exceed 6 gal. per sack of cement for any individual batch in localities where the concrete will be subjected to severe freezing and thawing conditions. In no case shall the mixing water exceed 6½ gal. per sack of cement for any individual batch. The cement content shall be not less than 5 sacks per cubic yard.

(*d*) Unless otherwise specified in the special provisions the following requirements shall govern the contractural relations concerning proportions based on design for minimum strength:

(1) Upon request, the Engineer will furnish prospective bidders with information as to the proportions by weight required for aggregates from established sources available for use on the project. This information will also include the grading of the aggregates used in determining these proportions.

(2) Promptly after receipt of notice of award of the contract, the Contractor shall furnish the Engineer with the location, or locations of the source or sources of aggregates which he proposes to use. The proportions will be designated by the Engineer. Except as hereinafter provided the designated proportions shall govern as long as materials are furnished from the sources designated and as long as they continue to meet the requirements specified.

(3) If, during the progress of the work, the contractor proposes to use aggregates from approved sources other than those originally designed, the Engineer will designate the new proportions to be used.

(4) If satisfactory plasticity and workability are not secured using the proportions and aggregates originally designated, the engineer may alter such proportions. If such alterations change the designated cement factor originally fixed by 2 per cent or less, no adjustment in the amount paid the contractor shall be made. If such alterations change the designated cement factor by more than 2 per cent, the source and quality of the aggregates remaining the same, payment shall be adjusted for or against the contractor in whatever amount the total cost of materials, f.o.b. contractor's material yard, has been increased or decreased by more than 2 per cent. The calculation of the amount of such increase or decrease shall be based upon the designated cement factor and not on count of bags of cement used or of the batches where bulk cement is used.

18. Proportions based on uniform cement content

(*a*) The proportions herein specified have been shown by experience to give satisfactory results for materials meeting the requirements of these specifications. They are so fixed as to produce concrete of constant

yield for all types of materials meeting the requirements of these specifications. The proportions of water, cement and saturated surface dry aggregates for the mixture shall be selected from Table 1, unless otherwise specified in the special provisions. To determine the pounds of saturated surface dry aggregate per bag of cement, multiply the factors given under Sand, Gravel, Crushed Stone, or slag by the bulk specific gravities of the aggregates to be used. Batch weights thus determined, shall be corrected to take into account the moisture condition of the aggregates as delivered to the measuring bin. If the aggregates contain more water than required for saturation the weight shall be increased accordingly; if they contain less water than required for saturation the weights shall be decreased accordingly.

(*b*) Mixture A shall be used when the concrete is to be compacted and finished with a standard finishing machine without vibration. Mixture B shall be used when the concrete is to be compacted and finished by vibration and it is desired to maintain the same water-cement ratio as when the standard finishing machine without vibration is used. Mixture C shall be used when the concrete is to be compacted and finished by vibration and where it is desired to maintain the same cement content as when the standard finishing machine is used.

(*c*) The ratio of fine to coarse aggregate may be adjusted, for different gradations within the limits of these specifications, to secure concrete of satisfactory plasticity and workability. During the progress of the work, the total weight of aggregate per bag of cement shall not be changed, except as necessary to compensate for differences in specific gravity, and except when concrete of satisfactory plasticity and workability cannot be made without exceeding the maximum net water content specified; under these latter circumstances the Engineer shall reduce the total weight of aggregate as computed from Table 1, by an amount sufficient to insure that the maximum net water content will not be exceeded. The Contractor shall not receive additional compensation for any extra cement which may be required by such adjustment.

MEASUREMENT AND HANDLING OF MATERIALS

19. Cement, aggregates, water

(*a*) Standard size sacks of cement as packed by the manufacturer shall be considered to weigh 94 lb. net. Bulk cement and cement from fractional sacks shall be weighed.

(*b*) When package cement is used, the cement shall be emptied from the sacks into the batch immediately prior to mixing. When bulk cement is used, satisfactory methods of handling and weighing shall be employed. A separate hopper shall always be so designed and operated that the

TABLE 1—PROPORTIONS BASED ON UNIFORM CEMENT CONTENT

Factors Used for Determining Weights of Saturated Surface-Dry Fine and Coarse Aggregate Per Bag (94 lb.) of Cement

Mixture	Cement Bags per Cubic Yard	Maximum Net Water Content Per Bag of Cement, Gal.	Slump in Inches	Factor (F) x bulk specific gravity = lbs. of saturated surface dry aggregate per Bag of Cement								
				When Gravel Is Used as Coarse Aggregate			When Crushed Stone Is Used as Coarse Aggregate			When Slag Is Used as Coarse Aggregate		
				Sand	Gravel		Sand	Crushed Stone		Sand	Slag	
					Fine Size	Coarse Size		Fine Size	Coarse Size		Fine Size	Coarse Size
				F	F	F	F	F	F	F	F	F
A	6.0	6.0	2–3	66	53	81	75	49	76	79	48	73
B	5.5	6.0	1–1½	66	64	96	75	60	91	79	59	88
C	6.0	5.5	1–1½	60	59	87	68	55	83	71	54	81

NOTE: Using the maximum allowable water content, the weights of Aggregates as determined by the above factors (when multiplied by the appropriate number of bags of cement per cubic yard plus the water and cement will produce a cubic yard of concrete. (If an amount of water less than the maximum is used the weights of aggregates will be adjusted to maintain the same minimum cement content as indicated.

quantity of cement for each batch will be maintained in a separate container so as not to come in direct contact with the aggregates.

(*c*) Aggregates shall be weighed. Each size of aggregate shall be weighed separately.

(*d*) The scales for weighing cement and aggregates shall be accurate within one-half of one per cent throughout the range of use. Operation shall be within a maximum allowable error of one per cent for cement and two per cent for aggregates.

(*e*) Mixing water may be measured by weight or by volume. The water measuring device shall be accurate to ½ of 1 per cent. Operation shall be required to be within a maximum allowable error of 1 per cent. When wash water is used as a portion of the mixing water for succeeding batches it shall be measured according to this requirement.

MIXING

20. Mixing at site or at central mixing plant

(*a*) The concrete shall be mixed in a batch mixer. When a drum mixer is used it shall conform to the requirements of the concrete mixer standards of the Mixer Manufacturers Bureau of the Associated General Contractors of America. If another type of mixer is used it shall be of a type satisfactory to the Engineer. The mixer shall be capable of combining the aggregates, cement and water into a thoroughly mixed and uniform mass within the specified time, and of discharging the mixture without segregation. Each batch of concrete shall be mixed for one minute or more after all materials, exclusive of the mixing water, are in the mixer drum. The batch shall be so charged into the mixer that some water will enter in advance of the cement and aggregate, and will continue to flow for a period which may extend to the end of the first one-third of the specified mixing time. The mixer shall rotate at the rate recommended by its manufacturer. Concrete shall be mixed only in quantities required for current use; any concrete which has set so that it cannot be placed properly shall not be used. Retempering of concrete which has partially set, by mixing with additional water, will not be permitted.

(*b*) The mixer shall be equipped with a suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked. Controls shall be so arranged that the water can be started only while the mixer is being charged and so as to lock automatically the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged after each charge has been mixed the required time. In case of mixing at the site suitable equipment for discharging and spreading

the concrete on the subgrade will be provided. The mixer shall be cleaned at suitable intervals. The pick-up and throw-over blades in the drum shall be replaced when they have lost ten (10) per cent of their depth. The volume of the mixed material per batch shall not exceed the manufacturer's normal rated capacity of the mixer, exclusive of the overload. The manufacturer shall install a plate upon the mixer stating the rated capacity and the recommended revolutions per minute.

21. Ready mixed concrete

Ready mixed concrete shall be mixed and transported in accordance with the current ASTM Specifications for Ready Mixed Concrete (Designation C 94).

HIGH EARLY STRENGTH CONCRETE

22. Methods of production

High early strength concrete shall be produced by one of the following methods, or any combination thereof, as specified in the special provisions.

(*a*) By the use of high-early strength portland cement Type III in lieu of normal portland cement (Type I or Type II).

(*b*) By the use of additional normal portland cement (Type I or Type II) in which case the total amount of cement shall not exceed seven bags of cement per cubic yard of concrete.

(*c*) By the use of calcium chloride as one of the ingredients of the concrete in an amount between one and two pounds of calcium chloride per sack of cement. Calcium chloride may be added either dry or in solution. When used in solution, it is convenient so to proportion the solution that one quart contains one pound of calcium chloride.

High early strength concrete shall meet the requirements of these specifications for portland cement concrete.

SUBGRADE PREPARATION

23. Grading stakes

Grading stakes shall be protected by the Contractor. Stakes which become disturbed shall be reset at the Contractor's expense.

24. Disposal of excavated material

Material removed in excavation shall be used in fills on the job, or otherwise disposed of as provided on the plans or in the special provisions. If excavation is in excess of fill, the excess material shall be disposed of by the Contractor at his expense.

25. Excavation below the subgrade

Where the work is in cut, if the Contractor excavates below the surface of the subgrade shown on the plans, he shall refill with approved material compacted as provided for filling and compaction at his expense.

26. Borrowed material

If filling is in excess of excavation, the contractor shall secure the necessary material from borrow pits indicated on the plans. If no such location is indicated or if the amount in the indicated location is insufficient he shall furnish material satisfactory to the Engineer. Payment shall be made for such material at an agreed-upon price per cubic yard.

27. Removal of sod

Any sod within the limits of the fill shall be completely removed to a depth of at least six inches.

28. Filling and compacting

All fills shall be made in layers not more than 9-in. thick which shall be thoroughly compacted by means of a smooth roller, pneumatic tired roller, or a tamping, or sheepsfoot roller. Smooth rollers shall be self-propelled and weigh not less than six tons nor less than 250 lb. per in. width of tread.

Pneumatic tired rollers shall be towed by mechanical equipment sufficiently powered to maintain an even rolling speed. The roller shall be capable of delivering at least 200 lb. per in. of tire tread, and an internal pressure of 4.5 psi shall be maintained.

Tamping or sheepsfoot rollers shall consist of metal rollers, drums or shells surmounted by metal studs with tamping feet projecting not less than 6 in. from the surface of the roller, drum or shell. The rollers shall be of such weight that the load upon each tooth, when any one row of teeth is supporting the whole weight of the roller, shall not be less than 1,250 lb. Each tamping roller shall consist of two sections and the length of each section shall not be less than four feet.

Inaccessible places, and places where rolling is not practicable on account of possible damage to subsurface structures, shall be thoroughly compacted with mechanical tampers capable of striking a blow equivalent to at least 250 lb. per sq. ft. The dead weight of any mechanical tamper shall be in excess of 40 lb. per sq. ft. of bearing surface. Payment for compacting layers in fills by rolling or tamping shall be included in the contract price per cu. yd. for "Grading."

29. Removal of unsuitable material

Fill material shall not contain trash, brick, broken concrete, tree roots, sod or cinders, and all unsuitable material shall be removed to a depth of at least 18 in. below the finished surface of the subgrade. The Contractor will be paid for this work at the prices bid for grading. and materials used. When unsuitable material is removed from the subgrade the surface shall be brought to the correct elevation with approved material placed and compacted as specified herein. Payment for this work will be made at the contract prices for grading and materials.

30. Removing and grubbing for stumps

Where fills higher than 10 ft. are to be made the ground shall be cleared of all underbrush and trees. Where fills are less than 10 ft. high, all underbrush, trees, tree stumps and saplings shall be grubbed and entirely removed.

31. Solid rock

Solid rock (boulders of one cubic yard or more in content, or ledges in their original bed) shall be removed six inches below grade and shall be paid for at the contract price per cubic yard of solid rock.

32. Sidewalk grading

Sidewalk grading included in the contract shall be done at the same time the roadway is rough graded.

33. Payment for grading

The contract prices for grading shall include the cost of excavating and filling, clearing and grubbing, excavating and removing unsuitable materials as required, shaping the subgrade to the prescribed lines and grades, sloping cuts, intersections and approaches, and rolling or tamping below the subgrade in the case of fills; and in connection with street construction, sloping and filling between the curb and sidewalk pavement, and grading and shaping of sidewalk spaces where shown on the plans. The work shall be estimated to the grades and slopes shown on the plans.

34. Rolling the subgrade

The subgrade shall be prepared by compacting with a self-propelled roller weighing not less than 6 tons. Places inaccessible to the roller shall be compacted with mechanical tampers capable of striking a blow equivalent to at least 250 lb. per sq. ft. The dead weight of any mechanical tamper shall be more than 40 lb. per sq. ft. of bearing surface. If the sub-grade is disturbed after acceptance it shall be reshaped and compacted without additional compensation. Payment for compacting subgrade shall be included in the contract prices for grading.

35. Wetting the subgrade

Concrete shall be placed on a moist subgrade except where subgrade paper is required by the special provisions. The subgrade shall be thoroughly wetted a sufficient time in advance of the placing of the concrete to insure that there will be no puddles or pockets of mud when the concrete is placed. The degree of saturation shall depend upon the characters of materials in the subgrade. The subgrade shall not be allowed to dry out before the concrete is placed.

36. Placing subgrade paper

When specified in the special provisions the subgrade after it has been shaped and compacted, shall be covered with subgrade paper. Adjacent

strips of paper shall lap 4 in. and ends shall lap 12 in. After being placed on the subgrade, the paper shall be kept reasonably intact and shall not be hauled over nor have unnesessary holes punched through it.

37. Testing and correcting subgrade

Immediately prior to placing concrete, the subgrade shall be tested for conformity with the cross section shown on the plans by means of an approved template riding on the side forms. If necessary, material shall be removed or added, as required, to bring all portions of the subgrade to the correct elevation. It shall then be thoroughly compacted and again tested with the template. Concrete shall not be placed on any portion of the subgrade which has not been tested for correct elevation.

FORMS

38. Material, dimensions, setting

Side forms shall be of metal of a cross section approved by the Engineer. The height of the forms shall be equal to the specified edge thickness of the concrete. Building up of forms shall not be permitted. Flexible or curved forms of proper radius shall be used for curves of 100 ft. radius or less. Forms shall be of ample strength, not less than $\frac{7}{32}$ in. thick. They shall be provided with adequate devices for secure setting so that when in place they shall withstand, without visible springing or settlement, the weight, impact and vibration of the finishing machine. In no case shall base width be less than 8 in. for forms 8 in. or more in height. The flange braces must extend outward on the base not less than two-thirds ($\frac{2}{3}$) the height of the form. The forms shall be free from warp, bends, or kinks. Any variation of the top of a form from its true plane when tested with a 10 ft. straight edge shall not be greater than $\frac{1}{8}$ in. in 10 ft. and any variation laterally when tested with a 10 ft. straight edge shall not be greater than $\frac{1}{4}$ in. in 10 ft.

39. Grade and alignment

The alignment and grade elevations of the forms shall be checked and the necessary corrections made by the contractor immediately before placing the concrete. When any form has been disturbed or any subgrade thereunder has become unstable, the form shall be reset and rechecked.

40. Base support

The subgrade under the forms shall be compacted and cut to grade so that the form when set shall be uniformly supported for its entire length and at the specified elevations. Subgrade found to be below established grade at the form line shall be filled to grade in lifts of $\frac{1}{2}$ in. or less for 18 in. on each side of the base of the form and thoroughly re-rolled or

tamped. Imperfections and variations above grade shall be corrected by tamping or by cutting as necessary. In exceptional cases the Engineer may require suitable stakes to be driven to the grade of the bottom of the forms to afford additional firmness.

41. Staking forms

Forms shall be staked with three or more pins for each 10 ft. section. A pin shall be placed adjacent to each side of every joint if required. Form sections shall be tightly joined by a locked joint free from play or movement in any direction.

42. Advanced setting

Where practicable forms shall be set for at least 300 ft. in advance of the point where concrete is being placed. Forms shall remain in place at least 12 hours after the concrete has been placed and shall be cleaned and oiled each time they are used.

INSTALLATION OF JOINTS AND REINFORCEMENT

43. General

(*a*) Longitudinal and transverse joints shall be constructed as shown on the plans.

(*b*) Transverse joints shall be expansion joints, contraction joints, or construction joints. Unless other locations are shown on the plans longitudinal joints shall be along or parallel to the center line of the pavement. Transverse joints shall be at right angles to the center line and shall extend the full width of the pavement. All joints shall be per pendicular to the finished grade of the pavement and, when tested with a 10 ft. metal straight-edge placed at right angles to the joint, the surfaces of adjacent slabs shall not vary from each other by more than $\frac{1}{8}$ in. Joints shall be finished as shown on the plans or as indicated in the special provisions.

(*c*) Especial care shall be taken to finish the subgrade accurately to the required cross-section, at the locations where transverse joints are to be installed. If, for any reason, the subgrade is trimmed too low, or if there are any open spaces beneath the joint, the joint material shall be removed, the subgrade backfilled and firmly tamped, and the joint reset.

(*d*) The edges of the pavement at joints shall be rounded to the radius shown on the plans. The joints shall be continuous from form to form, from joint to joint, or from form to edge of slab, as the case may require. The final horizontal position of transverse joints shall be at right angles to the center-line of the pavement on tangents and radially on curves. Longitudinal joints and reinforcement, except dowels, shall not extend through any transverse joints.

44. Longitudinal joints

(*a*) Tie bars shall be placed across longitudinal joints as shown on the plans. They shall be held in position, by chairs or other supports.

(*b*) Longitudinal "metal strip" joints shall be formed by installing a metal parting strip of the gage, shape and dimensions shown on the plans, to be left permanently in place. The metal strip shall be held securely in place, true to line and grade by suitable steel pins, which may be either channel shaped or round bars, spaced at intervals that average not less than 3 ft. and in no case more than 4 ft. The minimum length of metal strips shall be 10 ft. and adjoining sections shall be securely fastened by lapping and pinning, by means of a slip joint, or other approved method. The Contractor shall furnish an approved gage to ride on the side forms for checking the position of the parting strip before concrete is placed against it.

(*c*) Longitudinal "dummy" joints shall be formed by a groove or cleft in the top of the slab of the dimensions shown on the plans. The groove made in the soft concrete by a suitable tooling device, shall extend vertically downward from the surface to at least one third the depth of slab and shall be true to line. When the joint is of the poured type, the groove, as soon as possible after its completion, shall be cleaned and poured full of approved sealing material. When the joint is of the premolded type, strips of premolded filler shall be inserted in the groove immediately after it is formed.

(*d*) Longitudinal construction joints (i.e. joints between slabs placed separately) shall be formed as shown on the plans or as indicated in the special provisions. In forming these joints the contact edge (or edges) of the previously completed slab (or slabs) shall be painted with a heavy coat of bituminous material before the concrete of the adjacent slab is placed, unless a premolded joint filler is required and used between the slabs. In such cases the joint shall be trimmed and sealed as required for transverse expansion joints.

45. Expansion joints at structures

Expansion joints shall be formed about all structures and features projecting through, into, or against the pavement. Unless otherwise indicated on the plans, such joints shall be not less than ¼ in. thick and shall be of the premolded type.

46. Transverse premolded expansion joints

(*a*) Transverse joints using premolded expansion material shall be formed during the placing of the concrete. The methods of construction shall be such that the joints will extend to the full depth and width of a slab. The finished joint shall be true to the transverse line prescribed with an allowable variation of ¼ in. in the width of one traffic lane.

(*b*) Transverse premolded expansion joints shall be formed by securely staking an approved "installing bar" or installing device perpendicular to the proposed surface of the pavement. The installing bar shall be of substantial metal plate and shall have a length one-half inch less than the required width of the slab and shall be cut to the required depth and crown of the slab. It shall be securely staked in position so that the top edge, unless otherwise provided on the plans, will be uniformly ¼ in. below the pavement surface. The lower edge shall be cut to conform to the prescribed cross-section of the subgrade. The installing bar shall be slotted from the bottom as necessary to permit the installation of the required dowels. Suitable means shall be provided on the bar for facilitating its removal. Header boards, sheet metal holders or other devices used in lieu of the installing bar shall be subject to approval by the Engineer. The joints shall be protected against damage until they are installed in the work. Joints damaged during transportation, or by careless handling, or while in storage shall be replaced or repaired by the Contractor. Repaired joints shall not be used until they have been approved by the Engineer.

(*c*) The designated premolded joint filler shall be appropriately punched to the exact diameter and at the locations of the dowels. It shall be furnished in lengths equal to one-half the designated width of the slab. Where more than one section is used in a joint, they shall be securely laced or clipped together. The premolded joint filler shall be placed on the side of the "installing bar" nearest the mixer. The bottom edge of the filler shall extend downward to or slightly below the bottom of the slab, and the top edge, unless otherwise prescribed, shall be held about one-half inch below the surface of the pavement in order to allow the finishing operations to proceed continuously. The top edge of the filler shall be protected while the concrete is being placed, by a metal channel cap of at least ten gage material, having flanges not less than 1½ in. deep. The installing device may be designed with this cap self-contained.

47. Transverse metal strip joints

Transverse "metal strip" contraction joints shall be formed during the placing of the concrete by installing a metal parting strip to be left in place. This strip shall conform to the requirements for type, gage, shape, and dimensions shown on the plans or indicated in the special provisions and shall be securely staked in place. This strip shall be temporarily capped with a metal channel cap. The dowel assembly and the method of placing it shall be as indicated for transverse expansion joints, except that sleeves or caps on dowels will not be required. Concrete shall be deposited as for transverse expansion joints. After the longitudinal floating has been completed but before the surface is finished the metal cap shall be

removed and the concrete edged, and the joint sealed as required for expansion joints.

48. Transverse weakened plane joints

Transverse joints of the weakened plane or "dummy" type shall be constructed in a manner similar to that provided for dummy joints for longitudinal joints, except that such joints shall be provided with slip dowels or other load transfer devices of the type, size, and arrangement shown on the plans.

49. Transverse construction joints

Unless other prescribed joints occur at the same points, transverse "construction" joints shall be made at the end of each day's run or where interruption of more than 30 min. occurs in the concreting operations and the length of pavement laid from the last joint is more than 10 ft. Tie bars ½ in. in diameter and 3 ft. in length shall be placed at intervals of 2½ ft. across all transverse construction joints.

50. Transverse weakened plane contraction joints in pavement base

Concrete bases shall be provided with expansion and contraction joints as shown on the plans. Unless otherwise shown on the plans contraction joints in bases may be made by forming a cleft or groove in the upper portion of the slab either by installation of subgrade paper, or other approved water-proof material, or by means of an installing bar or device used to form the cleft or groove. The installing bar shall be designed to remain straight and true during operation. The groove shall extend across the full width of the slab in a straight line and shall extend vertically downward from the surface for at least one-half the depth of the slab. The groove shall be formed before the concrete has taken an initial set and in such a manner that the finished concrete base shall not be unduly disturbed. Reinforcement shall not extend across weakened plane joints.

Note: For bases other than those laid monolithicly (such as brick vibrated into fresh concrete) with the surfacing material, it is especially important to reduce the interval between joints by means of closely spaced contraction joints so as to control the amount of joint opening (which may extend through the surface course) due to shrinkage and contraction. Experience indicates that the best results are obtained when the spacing of such joints does not exceed 20 ft. Preferably the spacing should not exceed 15 ft.

51. Load transfer devices

(*a*) Dowel bars or some other approved type of load transfer device shall be placed across all transverse joints in the manner shown on the plans. They shall be placed at the approximate center of the slab depth. Dowel bars shall be held rigidly in horizontal and vertical alignment either by suitable holders to be left permanently in place, or they shall be installed as preassembled units wherein the dowels are held in proper

alignment by spacer bars welded or otherwise firmly secured to the fixed ends of the dowels. A tolerance of one in 36 from correct alignment for each dowel, and one in 48 from any adjacent dowel, will be permitted.

(*b*) Supporting stakes, when used, shall be located near the ends of the dowel bars. They shall not be attached to the spacer bars, nor shall they be placed to support the dowel bars between the spacer bar and the expansion joints. They shall be left permanently in place.

(*c*) The free end of each dowel bar shall be painted with one coat of red lead or basic sulphate blue lead paint at the site of the work. When the paint has dried, the free end of each bar shall be thoroughly coated with heavy oil immediately before it is placed in position. The free end of each bar for expansion joints shall be provided with a close fitting dowel sleeve.

(*d*) When a load transfer device other than a dowel bar is used it shall be of the type, size, and spacing shown on the plans, and shall be installed in a manner meeting the approval of the engineer.

52. Installation of joints

(*a*) If the paving mixer is operated from the shoulder, the joints shall be set immediately after the final testing of the subgrade. If the paving mixer is operated from the subgrade, the joints shall be set immediately after it moves forward, so as to permit as much time as possible for proper installation.

(*b*) The assembled joint shall be put in place on the prepared subgrade. It shall be placed at right angles to the centerline of the pavement. The top of the joint shall be set at the proper distance below the pavement surface and the elevation checked by a properly designed templet. On widened curves, the longitudinal center joint shall be placed so that it will be, as nearly as possible, equidistant from the edges of the slab. The joint shall be set to the required line and grade and shall be securely held in the required position by stakes or an approved installing device or both during the placing and finishing of the concrete. Joints shall be installed so that the concrete pressure will not disturb their alignment. The joints shall be vertical and no joint shall deviate more than one-fourth inch in horizontal alignment either way from a straight line. If joints are constructed in sections, there shall be no offsets between adjacent units. Dowel bars shall be checked for exact position and alignment as soon as the joint is staked in place on the subgrade and the joint shall be tested to determine whether it is firmly supported. Any joint not firmly supported shall be reset.

53. Placing reinforcement

(*a*) Reinforcing steel, when placed in the work, shall be free from

dust, dirt, scale, or other foreign matter, and rust of such degree of development as to impair bond of the steel with the concrete.

(*b*) When bar assemblies are shown on the plans, the reinforcement bars shall be firmly fastened together at all intersections. Adjacent ends shall lap not less than 30 diameters.

(*c*) Where bars are fabricated into mat form by positive welding at all intersections, the lap may be of such length as will permit cross bars to overlap each other by at least 2 in. The same requirements apply to end laps and side laps.

(*d*) Steel fabric sheets shall be lapped and shall clear all edges of the slab, as shown on the plans. Where steel fabric is used, the concrete shall be struck off by means of a template at the indicated depth of the reinforcing below the finished surface of the slab. The fabric reinforcement shall then be placed directly upon the concrete after which the balance of the required concrete shall be placed above it.

(*e*) Where sheets of wire mesh or bar mats are required to be lapped, adjacent sheets or mats shall be tied so as to maintain the required lap during placement of the concrete.

(*f*) If shown on the plans, marginal bars shall be placed on chairs or supports, to insure that the bars will be in correct position after the concrete is placed. They shall be given one coat of red lead or of basic sulphate blue lead paint. At the time the marginal bars are placed, the paint shall be dry and hard. After being placed in position, the marginal bars shall be coated with heavy oil.

(*g*) Displacement of reinforcement during concreting operations shall be prevented.

PLACING AND FINISHING CONCRETE

54. General

(*a*) Concrete shall be distributed to such depth, above grade, that when consolidated and finished, the specified slab thickness will be obtained at all points and the surface will not be below the grade specified for the finished surface at any point.

(*b*) The concrete shall be deposited on the subgrade in such manner as to require as little rehandling as possible. It shall be thoroughly consolidated against and along the faces of the forms. Necessary hand spreading shall be done with shovels, not with rakes. Workmen shall not be allowed to walk in the concrete with boots or shoes covered with earth or other foreign substances.

(*c*) Concrete shall be placed only on subgrade which has been prepared as specified and approved. At all times during operations, at least 300

ft. of subgrade shall have been prepared ahead of the mixer. Concrete shall not be placed on a frozen subgrade. No concrete shall be placed around manholes or other structures until they have been brought to the required grade and alignment.

(*d*) Pavement less than 30 ft. wide may be constructed either to its full width in a single construction operation or in lanes, unless one or the other method is expressly stipulated on the plans. Pavement more than 30 ft. wide shall not be constructed in a single operation. When pavement is constructed in separate lanes, the junction line shall not deviate from the true line shown on the plans by more than ½ in. at any point.

(*e*) Retempering concrete by adding water or by other means will not be permitted.

55. Placing and finishing concrete at joints

(*a*) Concrete shall be deposited on the subgrade as near to the expansion and contraction joints as possible without touching them. It shall then be shoveled against both sides of the joint simultaneously, maintaining equal pressure on both sides. It shall be deposited to a height of approximately two inches more than the depth of the joint, care being taken that it is worked under the load transfer devices. The concrete shall not be dumped from the discharge bucket of the mixer directly upon or against the joints, nor shall it be shoveled or dropped directly on top of the load transfer devices. In placing the concrete, against expansion and contraction joints and in operating a vibrator adjacent to them, workmen shall avoid stepping upon or disturbing in any way the joints or load transfer devices, either before or after they are covered with concrete.

(*b*) The concrete adjacent to the joint shall be compacted with a vibrator inserted in the concrete and worked along the entire length and on both sides of the joint. The vibrator shall not come in contact with the joint, the load transfer devices, or the subgrade. If any of the dowel bars are displaced, they shall be realigned before the finishing machine passes over them.

(*c*) After the concrete has been vibrated, the finishing machine shall be moved forward until the front screed is approximately eight inches from the joint. Segregated coarse aggregate shall be removed from both sides of and off of the joint. The screed shall be lifted and brought directly above the joint, set upon it, and the forward motion of the finishing machine shall be resumed. When the second screed is close enough to permit the excess mortar in front of it to flow over the joint, it shall be lifted and carried over the joint. Thereafter, the finishing machine may be run over the joint without lifting the screeds, provided there is no

segregated coarse aggregate immediately between the joint and the screed or on top of the joint.

(*d*) After the concrete has been placed on both sides of the joint and struck off, the installing bar or channel cap shall be slowly and carefully withdrawn leaving the premolded filler in place. After the installing bar or channel cap is completely withdrawn the concrete shall be carefully spaded and additional freshly mixed concrete worked into any depressions left by the removal of the installing bar. The installing bar shall be cleaned and recoiled prior to each installation of a joint.

Immediately after all finishing operations, including brooming, have been completed and before the concrete has taken its initial set, it shall be edged adjacent to all expansion and contraction joints. Care shall be used to remove any concrete which may be over the premolded joint material. The edging tool shall be so manipulated that a well-defined, continuous radius is produced and a smooth, dense mortar finish obtained. The edging tool shall not be tilted while being manipulated.

After the removal of the side forms, the ends of premolded transverse joints at the edges of the pavement shall be carefully opened for the entire depth of the slab, and in the case of air cushion joints any concrete that has been deposited over the end closure shall be removed, care being taken not to injure the ends of the joint. After the curing period and before the pavement is opened to traffic, premolded joints shall be sealed or topped out, leaving a neat uniform strip of an approved filler material.

56. Finishing methods

(*a*) *Machine finishing.* The concrete as soon as placed shall be struck off and screeded by an approved finishing machine to the crown and cross-section shown on the plans and to an elevation slightly above grade so that when properly consolidated and finished the surface of the pavement will be at the exact grade elevation indicated on the plans and free from porous places.

The finishing machine shall be of the screeding and troweling type, equipped with two independently operated screeds, designed and operated both to strike off and to consolidate. The machine shall go over each area of pavement as many times and at such intervals as is necessary to give the proper compaction and to leave a surface of uniform texture, true to grade and crown. Excessive operation over a given area shall be avoided. The last trip for a given area shall be a continuous run of at least 40 ft. The tops of the forms shall be kept clean by an effective device attached to the machine and the travel of the machine on the forms shall be maintained true without lift, wobbling, or other variation tending to affect the precision of finish. The finishing machine shall

be of ample strength to withstand severe use and shall be fully and accurately adjustable for loss of crown or other derangement due to wear.

During the first pass of the finishing machine a uniform ridge of concrete at least 3 in. deep shall be maintained ahead of the front screed for its entire length. Except when making a construction joint, the finishing machine shall not be operated beyond that point where the above described surplus can be maintained ahead of the front screed.

(*b*) *Hand finishing.* Where hand finishing is permitted the concrete shall be struck off and consolidated by means of a metal shod screed to the crown and cross-section shown on the plane and to such an elevation above grade that when consolidated and finished, the surface of the pavement will be at the required elevation. The screed shall be moved forward with a combined longitudinal and transverse motion, moving always in the direction in which the work is progressing and so manipulated that neither end is raised from the side forms during the strike-off process.

(*c*) *Vibrated finishing.* When vibrated finishing is required, by the special provisions, the finishing machine shall be equipped for applying high frequency vibration to the upper surface of the concrete in accordance with the requirements for either of the following types. The particular type selected by the contractor shall be subject to approval of the Engineer. The vibratory units shall be synchronized and shall operate at a frequency of not less than 3,500 cycles per minute.

1. Vibrating units mounted directly on the screeds. This type shall consist of two independently operated screeds. The front screed shall be equipped with not less than one vibrating unit for each 7½ ft. length or portion thereof of vibratory screed. The front screed shall be not less than 15 in. wide and shall be equipped with a bull-nose front edge having a radius of not less than 2 in.

2. Vibrating pan mounted independently of the screeds. This type shall consist of two independently operated screeds together with an independently operated vibratory pan (or pans). The pan shall be mounted in such a manner as not to come in contact with the forms. It shall rest directly on the concrete and shall be so adjusted as to produce uniform vibration over the entire width of pavement surface. The pan shall be equipped with not less than one unit for each 7½ feet of length or portion thereof of vibrating pan. The screeds shall be capable of operating in a position that will strike off the concrete at a sufficient height above the top of the forms to allow for proper compaction with the vibratory pan.

3. Internal vibrators. Internal vibrators may be approved for use with the standard finishing machines if specified in the special provisions.

(*d*) *Floating.* As soon as possible, after the concrete has been struck off and consolidated, it shall be further smoothed and consolidated by means of a longitudinal float of a suitable design approved by the Engineer. In this operation the longitudinal float shall be worked with a sawing motion, while held in a floating position parallel to the road centerline and passed gradually from one side of the pavement to the other. Movements ahead along the centerline of the road shall be in successive advances of not more than one-half the length of the float.

(*e*) *Straightedging.* After the longitudinal floating has been completed and the excess water removed, but while the concrete is still plastic, the slab surface shall be tested for trueness with a 10 ft. straightedge swung from handles 3 ft. longer than one-half the width of the slab. The straightedge shall be held in successive positions parallel to the road centerline in contact with the surface and the whole area gone over from one side of the slab to the other. Advance along the road shall be in successive stages of not more than one-half length of the straightedge. Any depressions found shall be filled immediately with freshly mixed concrete, struck off, consolidated and refinished. High areas shall be cut down and refinished. The straightedge testing and refloating shall continue until the entire surface is found to be free from observable departures from the straightedge and the slab has the required grade and crown.

(*f*) *Belting.* After straightedging when most of the water sheen has disappeared and just before the concrete becomes non-plastic, the surface shall be belted with a two-ply canvas belt not less than 8 in. wide and at least 3 ft. longer than the width of the slab, or with an approved wooden belt. Hand belts shall have suitable handles to permit controlled uniform manipulation. The belt shall be operated with short strokes transverse to the road centerline and with a rapid advance parallel to the centerline.

(*g*) *Brooming.* As soon after belting and after the surplus water has risen to the surface, the pavement shall be given a broom finish with an approved steel or fiber broom, not less than 18 in. wide. The broom shall be pulled gently over the surface of the pavement from edge to edge. Adjacent strokes shall be slightly overlapped. Brooming shall be perpendicular to the center line of the pavement and so executed that the corrugations thus produced will be uniform in character and width, and not more than $\frac{1}{8}$ in. deep. The broomed surface shall be free from porous spots, irregularities, depressions and small pockets or rough spots such as may be caused by accidentally disturbing particles of coarse aggregate, embodied near the surface. Brooming will not be required for concrete bases.

(*h*) *Edging.* After belting and brooming have been completed, but before the concrete has taken its initial set, the edges of the slab shall be carefully finished with an edger of the radius required by the plans.

(*i*) *Final surface test.* The contractor will be held responsible for the correct alignment, grade, and contour specified. Any spots higher than ⅛ in. and not higher than ¼ in. for concrete pavements and higher than ¼ in. for concrete bases but not higher than ⅜ in. above the correct surface as shown by the 10 ft. straightedge, shall be ground to the required surface by the contractor at his own expense. Where deviation exceeds the foregoing limits the pavement slab shall be removed and replaced by the contractor at his own expense, as directed by the engineer.

57. Cold weather concreting

(*a*) Except by specific written authorization, concrete placing shall cease when the descending air temperature in the shade and away from artificial heat falls below 40 F. It shall not be resumed until the ascending air temperature in the shade and away from artificial heat rises to 35 F.

(*b*) When concreting is permitted during cold weather the temperature of the mixed concrete shall be not less than 60 F. nor more than 100 F. at the time of placing in the forms. The aggregates or water or both may be heated. The aggregates may be heated by steam or dry heat prior to being placed in the mixer. The water shall not be hotter than 175 F.; aggregates shall not be used which are hotter than 150 F.

(*c*) When concrete is being placed during cold weather and the air temperature may be expected to drop below 35 F., a supply of straw, hay grass or other suitable blanketing material shall be provided along the line of the work. At any time when the air temperature may be expected to reach the freezing point during the day or night the material so provided shall be spread over the concrete to a sufficient depth to prevent freezing of the concrete. Such protection shall be maintained for at least five days. If required by the Engineer, concrete less than 24 hours old shall also be covered by approved canvas or similar enclosures and devices capable of maintaining the temperature within the concrete at 50 F. or higher. Concrete injured by frost action shall be removed and replaced at the Contractor's expense.

58. Lighting conditions

When the natural light is insufficient for proper work, operations shall cease unless approved artificial lighting is provided.

CURING

59. Preliminary curing period

The concrete shall be covered with two thicknesses of wet burlap weighing not less than 7 oz. per sq. yd., cotton mats, or other approved material of highly absorptive quality, immediately after final finishing of the pavement surface. The material shall be kept saturated by spraying and shall remain in place for at least 12 hours.

60. Final curing

(*a*) For completion of the curing the burlap. cotton or other mats may be left in place and kept saturated for 72 hours, or they may be removed at the end of the preliminary curing period and the concrete surface protected by keeping the surface covered with water (ponding) for three days after placing of the concrete; or by a one-inch layer of thoroughly wet earth, or sand, or a six inch layer of thoroughly wet straw, hay or similar material maintained on the surface of the concrete and kept thoroughly wet for three days after placing of the concrete. Other methods of final curing may be used if approved by the Engineer. The sides of concrete slabs exposed by the removal of forms shall be protected immediately after the removal of the forms in such a manner as to provide these surfaces with a curing treatment equivalent to that provided by the method prescribed for the surface.

(*b*) Forms shall not be removed from freshly placed concrete until it has set for at least 12 hours.

MISCELLANEOUS

61. Pavement thickness

(*a*) The thickness of the pavement shall be determined by measuring the lengths of drilled concrete cores according to the procedure of the current ASTM Tentative Method C-174-42T. At such points as the Engineer may select, in each 1,000 lin. ft. of pavement, two or more cores shall be taken and measured. The average thickness of each mile of slab, or fractional mile, will be determined from these measurements.

(*b*) Pavement of which the average thickness is within $\frac{1}{8}$ in. of the thickness required by the typical cross-section shown on the plans, will be accepted and paid for at the contract price.

(*c*) For pavement, the average thickness of which is less than the thickness shown on the plans by more than $\frac{1}{8}$ in. but by less than $\frac{1}{2}$ in., an adjusted unit price shall be used in payment. This price shall bear the same ratio to the contract unit price as the square of the average thickness of the slab bears to the square of the thickness specified on the plans.

(*d*) Payment will not be made for pavement which is found deficient in thickness by ½ in. or more. Such pavement shall be removed and replaced by the contractor with pavement of the specified thickness at his expense. When the measurement of any core indicates that the slab is deficient in thickness by ½ in. or more, determination shall be made of the thickness of transverse sections of the slab at 25 ft. intervals set off along the center line of the road in each direction from the affected location until a transverse section of the slab is found which is not deficient in thickness by as much as ½ in. The area of pavement for which no payment will be made shall be the product of the width of pavement multiplied by the distance along the centerline of the road between the transverse sections found not deficient in thickness by as much as ½ in. If the Contractor is not satisfied he may request additional cores and measurements. Such measurement shall be made at intervals of not less than 200 ft. The cost of additional cores and measurements shall be deducted from any sums due the Contractor unless the measurements indicate that the slab within the area in question is of specified thickness.

(*e*) No additional payment over the unit contract price bid will be made for pavement found to be thicker than the specified amount.

62. Protection of finished pavement

(*a*) The Contractor shall erect and maintain suitable barricades and, when required in the special provisions, shall employ watchmen to exclude traffic from the newly constructed pavement until open for use. These barriers shall be so arranged as not to interfere with public traffic on any lane intended to be kept open, and necessary signs and lights shall be maintained by the Contractor clearly indicating any lanes open to the public. Where, as shown on the plans or indicated in the special provisions, it is necessary to provide for traffic across the pavement, the Contractor shall at his expense construct suitable and substantial crossings to bridge over the concrete.

(*b*) Any part of the pavement damaged by traffic or other causes prior to its final acceptance shall be repaired or replaced by and at the expense of the contractor in a manner satisfactory to the Engineer. The Contractor shall protect the pavement against both public traffic and traffic of his employees and agents.

63. Public use of thoroughfare

(*a*) Normal, unimpeded use of the thoroughfare of which the proposed pavement is to be a unit, is of value to the public. It is, therefore, mutually understood, that for the sections of the thoroughfare identified on the plans as requiring special traffic handling and for the distances

stated thereon, surfaced road lanes as indicated shall be made available by the Contractor for unimpeded public traffic at all times, and maintained in proper condition throughout the construction period. These lanes shall be of the clear widths stated on the plans, and shall be kept entirely free from encroachment by equipment of the Contractor, by workmen or employees of the Contractor or by storage or transportation of materials intended for the work.

(*b*) The scheme and sequence of construction of the several lanes, slabs and sections of pavement, including the sequence of the shifting of public lanes during progress of construction, shall be as given on the plans or as described in the special provisions.

(*c*) Where the edge of any stipulated public traffic lane is contiguous to an edge of the slab or lane being placed, the Contractor shall provide, erect and subsequently remove, a substantial temporary guard fence, as shown on the plans along the prescribed dividing line, which shall be maintained there until the slab is opened to traffic. The Contractor's plan of operation shall be such as to obviate any need for encroachment on the public traffic lane or lanes. Where so shown on the plans special lanes for the Contractor's trucks and similar vehicles shall be provided, separate from and not interfering with the prescribed traffic lanes. Where the clearance between public traffic lanes and the Contractor's operating equipment is restricted, special delivering equipment may be necessary, designed to deliver and depart within the width of the slab actually being placed without encroaching any public lane.

(*d*) Except where a special bid price for "traffic handling" is required in the proposal and in the special provisions, all cost of handling and protecting traffic, of special equipment, of temporary road surfacing and its maintenance, of temporary guard fences and of other things to be provided or to be done under this paragraph, shall be at the expense of the Contractor.

64. Opening to traffic

Traffic will ordinarily be excluded from the newly constructed pavement for 14 days after the concrete is placed and may be excluded for a longer period if cross-bending tests indicate its advisability. Cross-bending test specimens, prepared at regular intervals from the concrete as it comes from the mixer and cured under the same temperature, moisture and climatic conditions as the corresponding slabs of pavement, shall be employed as a means of fixing the time of opening the pavement to traffic. These beams shall be made and tested in accordance with Section 13. When tests of these specimens indicate that the corresponding pavement has attained a modulus of rupture of at least 450 psi. the pavement shall be cleaned, the joints filled and trimmed and the pavement

opened to traffic. If this strength is not attained within 14 days, the Engineer may open the pavement to traffic at his discretion. The joint or line of separation between adjacent strips or slabs of concrete, when the pavement is constructed in strips or slabs shall be cleaned and filled with an approved sealing material.

[To facilitate selective distribution, **separate prints** of this ACI Standard (title 41-28) are currently available from ACI at 25 cents each—quantity quotations on request.]

Title 41-28 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 41**

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Vol. 16 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1945**

ACI Standard

Specification for Cast Stone (ACI 704-44)*

Reported by ACI Committee 704

C. G. WALKER
Chairman

M. A. ARNOLD
OTTO BUEHNER
LOUIS GUZZETI
N. D. HARTER
JOHN TUCKER, JR.

The following proposed specification for cast stone is intended to supercede the specification tentatively adopted by the American Concrete Institute in February, 1929 (P3-A-29T) and which has since been generally recognized and used commercially as a standard for the physical quality of cast stone. The principal changes which have been made are an increase in compressive strength requirements from 5,000 psi. to 6,500 psi. and a reduction in permissible absorption for 7 per cent to 6 per cent. The absorption period has been increased from 24 to 48 hours, and the lower limit of 3 per cent on absorption has been eliminated.

I. GENERAL

1. The term "cast stone" shall be understood to mean a building stone manufactured from portland cement concrete, precast and set in place as trim or facing on or in buildings and other structures.

2. The minimum compressive strength of cast stone when delivered to the building site shall be 6,500 psi. when tested as 2 by 2-in. cylinders or 2 by 2-in. cubes in the manner hereinafter specified.

3. The average water absorption of cast stone when delivered to the building site shall be not more than 6 per cent by dry weight of the specimens when tested as 2 by 2-in. cylinders or 2 by 2-in. cubes in the manner hereinafter specified.

*Adopted as a Standard of the American Concrete Institute at its 40th Annual Convention March 2, 1944; ratified by Letter Ballot of the ACI Membership July 28, 1944.

4. All aggregate used in the manufacture of cast stone shall be of known durable quality.

II. SELECTION OF SPECIMENS FOR TESTING

5. Specimens for both compression and absorption tests shall be cut from stone as delivered to the job or from regular stock in the yard. Samples from which specimens will be cut shall be selected by the purchaser or his representative.

6. Specimens of faced cast stone for compression tests shall be cut in such a manner that the specimens are composed of approximately one-half of facing and one-half of backing material. Specimens shall be tested in the position in which the cast stone is laid in the wall.

III. METHODS OF TESTING

7. Not less than 3 and preferably 5 specimens shall be required for each test.

8. Specimens for absorption test shall be dried at a temperature between 215 and 225 F. until the loss in weight is not more than 0.1 per cent in 24 hours of drying. They shall be allowed to cool to room temperature, weighted and then submerged in water at a temperature between 60 and 80 F. for 48 hours. The specimens shall then be removed from the water, the surface water wiped off with a damp cloth and the specimens weighed. The percentage of absorption is the difference in weight divided by the dry weight of the specimen and multiplied by 100.

9. Specimens for strength test shall be dried at a temperature between 215 and 225 F. until the loss in weight is not more than 0.1 per cent in 24 hours of drying.

10. If bearing surfaces of specimens for strength tests are not smooth, they shall be made so by grinding. If they cannot be ground to a smooth surface they shall be capped with a mixture of ½ part portland cement and ½ part plaster of paris, which shall be allowed to harden at least 5 hours before the test. The cap shall be formed by spreading the capping material upon a plate glass and pressing the specimen firmly on it making the cap as thin as possible. On specimens that are to be capped, all surfaces which are likely to come into contact with the capping material shall be shellacked and then allowed to dry before the capping is applied.

11. Load shall be applied through a spherical bearing block placed on top of the specimen in a vertical testing machine. The dimensions of the bearing block shall be the same, or slightly larger, than those of the test specimen.

12. Load shall be applied uniformly and without shock. The speed of the moving head of the machine shall be not more than .05 in. per min. when the machine is running idle.

13. Specimens shall be loaded to failure and the unit compressive strength calculated in pounds per square inch. The type of failure and appearance of specimen shall be noted.

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Vol. 16 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1945

Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1944 to June 1945.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Precast Concrete Irrigation Pipe Jacked under Railroad Trucks (41-167)

By PAUL A. JONES*

The Buffalo Rapids Project of the Bureau of Reclamation was confronted with the problem of constructing ten canal laterals across a railroad right-of-way which divided an irrigable area. The conduits ranged from 18 in. to 42 in. in diameter. Inasmuch as the estimated cost of approximately $1,000 per structure for track false work and placing monolithic structures or precast pipe in open trenches was much in excess of the funds available it was necessary to devise some other type of construction.

Of the types of construction investigated the jacking method of installing precast pipe offered a practical and economical solution if difficulties in providing an adequate seal for the joints could be overcome. This problem was solved by the adoption of a manufactured rubber gasket with projecting lips which permitted movement at the joint without losing the effectiveness of the seal.

*Construction Engineer, Bureau of Reclamation, Buffalo Rapids Project, Montana.

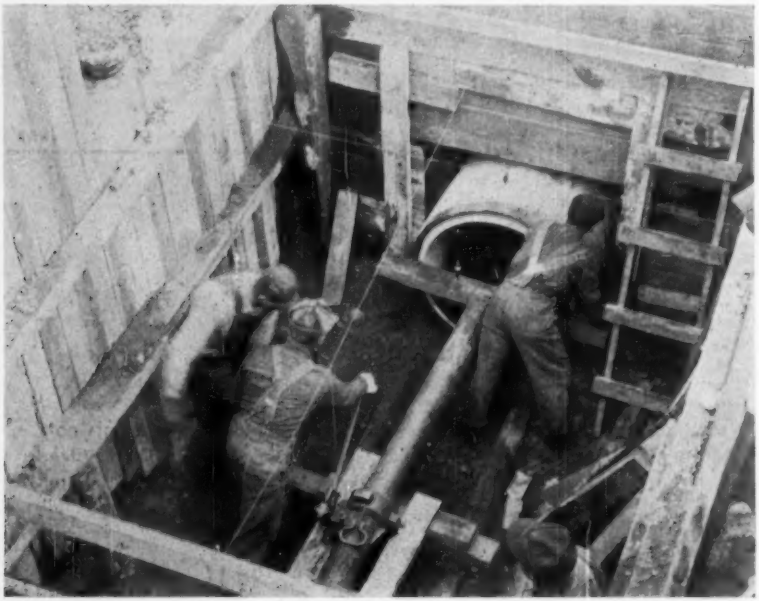

Fig. 1—Jacking 36-in. diameter precast concrete pipe under railroad tracks

The concrete pipe used was extra-strength pipe purchased under ASTM Specifications C76-37, the required concrete strength being 4,500 psi. The specifications stipulated that the tongued and grooved joints have an annualar clearance between abutting ends of not more than 3/16-in. and a variation between the actual and specified internal and external diameter of not more than ½ of one percent.

Fig. 1 shows a 36-in. diameter pipe being jacked under the roadbed. The installation of this and the other crossings was accomplished using the following procedure. A minimum working area of 10 + 12 ft. was excavated as close to the tracks as safety permitted. This hole was shored in loose ground. Screeds for supporting and alining the pipe were then laid to line and grade on a thoroughly compacted subgrade and securely anchored to the sides of the excavated working area. A length of pipe was placed on the screeds and forced against the excavated face of the roadbed with a 50-ton jack. When available, a ⅜-cu.-yd. dragline was used to handle the pipe but for most of the crossings the pipe was placed in jacking position by the use of a winch, block and tackle, and snubbing posts. Equipment used in excavating from the inside of 18-in. diameter pipe consisted of a 10-inch post hole digger, a specially designed scoop shaped to fit the inside of the pipe, and a wide flat steel plate mounted on roller skate wheels on which a man moved in and out of the pipe. After the excavation reached a distance of

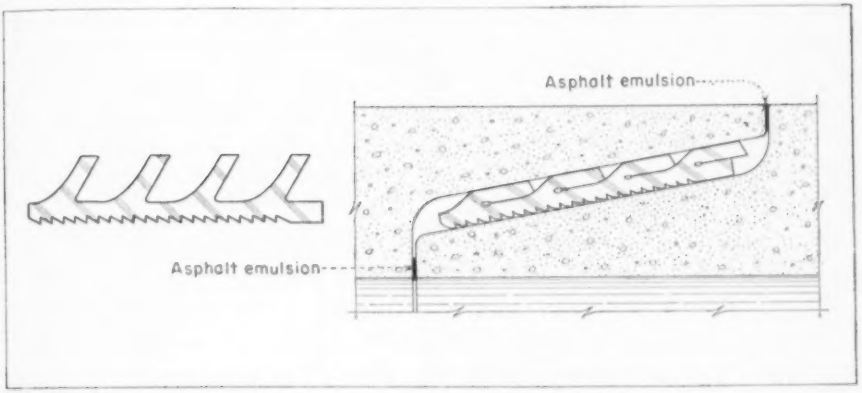

Fig. 2—Pipe joint detail: at left, section through rubber gasket; at right, section through joint of 36-in. diameter pipe.

about 16 feet a hole was augered completely through for ventilation. At this point a flat pan mounted on roller skate wheels was used to transport the scoop into the pipe and after two scoopfuls of excavation were placed in the pan it was removed and unloaded. Excavation for pipe larger than 18 in. was performed by a man working within the pipe, using a short-handled shovel. A box mounted on three small steel wheels was used to facilitate removal of the material thus excavated. To maintain accurate alinement and grade of the larger crossings, the bottom 90 degrees of the excavation was carefully trimmed, and checked with a sheet-iron template, to fit the outside of the pipe. A clearance of approximately one inch was left for the remaining circumference. The pipe was jacked at intervals of about one foot and when jacking had proceeded to the end of a pipe section, a rubber seal and another length of pipe was added. A construction crew consisting of one subforeman and four laborers carried on this work efficiently.

When installing a rubber gasket on the tongued end of the pipe, rubber cement was applied to the inner side of the gasket and a special lubricant was applied to the outer side, in accordance with the recommendations of the manufacturer. To provide protection for the rubber seal, asphalt emulsion was applied to the pipe shoulders where abutting ends come in contact. Care was exercised to prevent the emulsion from contacting the rubber seal. A section of the rubber seal and of a completed joint is shown in Fig. 2.

The length of pipe jacked for the various crossings varied from 20 feet to 52 ft. The longest installation was 30 in. in diameter and was under the main line and two side tracks. No difficulty was experienced in jacking this length with a single 50-ton jack. However, indications were that any additional length would have strained the jack.

One of the difficulties encountered was excavating in the cramped space within a 24-in. pipe. The smallest man available had to work the material back with his feet. This work was not excessive for the 20 ft. of 24-in. pipe installed. However, longer crossings would entail considerable effort and reduce progress materially. It is probable that the procedure later developed for 18-in. diameter pipe would have been satisfactory in excavating for 24-in. pipe.

Two crossings using 36-in.-diameter pipe were excavated through gravel. In this loose material the pipe had a tendency to "dig in" as jacking progressed. To combat this situation a "sled" bent to fit the outside curvature of the pipe and curved upward at the forward end was installed on the front end of the pipe. The arrangement worked satisfactorily. In excavating loose material, use was made of a protective shield, fastened to the upper part of the pipe. This protected the worker from falling rocks and maintained a sloping face the toe of which was beyond the end of the pipe. Between working shifts, a removable bulkhead was fitted into the forward end of the pipe or into the excavation to reduce the likelihood of caving.

Frequent passage of trains caused considerable vibration and tended to enlarge the excavated openings. Such caving, if excessive, resulted in settlement of the tracks. This settlement was particularly noticeable where the clearance between the pipe and rails was small. Plans drawn for the railroad crossings originally required 3-ft. clearance between top of pipe and base of rail. No trouble was experienced in jacking 24-in. pipe under these conditions. However, one 30-in. crossing was attempted with 3-ft. clearance and under heavy freight traffic the excavation could not be held in place. When work on the crossing was interrupted and left for a few days, it was difficult to jack the pipe farther. It was decided that the arching effect of 3 ft. of cover was insufficient for pipe 30 in. or larger. When clearance between top of pipe and base of rail was increased to 5 ft., the vibration from train passage had little or no effect.

Following is an estimate of the cost of installing 30-in. pipe crossings. This cost, except for the difference in cost of the pipe itself, is a close approximation for the 24-in. and 36-in. sizes as increased handling and excavation costs for the larger size are very nearly offset by the more difficult excavation for the smaller size.

	Cost per foot of pipe jacked
Work preliminary to jacking	$ 1.21
Excavation and jacking pipe	4.82
Pipe and joint materials	4.76
Engineering and supervision	1.08
	$11.87

Relatively large savings were effected by jacking the pipe under the tracks and the quality of the completed work is equal to that obtained by installation of pipe in an open trench. It is believed that the life of the rubber joints as installed will compare favorably with that of grouted joints, and that more flexibility under load and settlement is provided.

Setting Heavy Machinery on Concrete Bases (41-164)*

The question and discussion under this heading in February seem to have prompted further discussion of the subject of machinery bases and further inquiry on special problems. For instance a construction engineer with a large industrial corporation writes:

Q—In connection with a large compressor installation, which we made, the unbalanced load of the compressors caused considerable vibration to the entire concrete mat under the foundations. Some time after the original foundations were installed an additional compressor was added, and the concrete mat was joined to the existing mat with the reinforcing dowel bars extending through from the original concrete structure into the new. The question has now arisen whether there is enough movement in the concrete to prevent proper bond between the reinforcing rods and the concrete, during the period of the initial set.

From an ACI Member to whom this inquiry was referred we have this:

A—The amount of bond secured around an embedded bar, vibrated while the concrete is setting, depends on the character of the vibration of the bar. We have some unpublished tests of the results of vibrating smooth ¾-in. bars in 6- by 12-in. concrete cylinders for 30 seconds at 15-minute intervals for periods of 1, 3, 5, and 7 hours. Some bars were vibrated in a vertical position, some horizontal. At 5 and 7 hours the mortar was still plastic around the bars although surrounding concrete had hardened. Conclusions were that such vibration materially increased the bond over that obtained under ordinary conditions and the beneficial results of the vibration were obtained whether the bars were vibrated in a vertical or horizontal position. Presumably this test does not quite represent the case reported because vibration did not continue until the mortar did not again become plastic around the bar when vibrated. Also this was intermittent, not continuous, vibration.

The quality of bond secured around a vibrating bar will depend also on the distance from the source of vibration and the depth of embedment. Also it seems to be a matter of amplitude. Although vibrating concrete will harden, if there is appreciable amplitude in a vibrating

*ACI JOURNAL, Feb. 1945, p. 360 (JPP 41-164)

bar in the concrete, it is conceivable that, as the concrete passes from the plastic state, the vibrating bar could have sufficient amplitude to press the weak concrete away from its normal position and prevent contact and bond. This may well be the case close to the edge of the new foundation where the dowels enter. Depending on the length of the dowels, surrounding concrete will dampen the vibration to the point where bond will be secured without question, particularly if the unvibrated end is fixed. There is some discussion of this point in the ACI JOURNAL for January 1945, pages 163-5.*

Probably under horizontal bars, especially if of square section laid flat, vibration will at first aggravate and then tend to dissipate the accumulation of bleeding water under the bar that would tend to impair bond on the underside of the bar.

As we understand the case, the compressor on one foundation continued running with resultant vibration of protruding dowel bars during the setting of concrete in an adjacent foundation in which the dowels were also embedded. It would be of interest to know if the question now raised is the result of some adverse structural evidence or merely academic. Either way it is a good question and not an uncommon one. Would it be practicable to answer this question positively with some drill cores?

By DOUGLAS S. LAIDLOW†

I must get into the discussion, with a different slant from that of Messrs. Tyler and Lewis, for the practice of placing grout under machinery and column bases has long seemed to me to be one of the hoary blunders of our profession.

See what happens: The manufacturer of a machine builds up an expensive base, either by casting or welding, and then takes the trouble to machine the under side of it truly parallel to the horizontal axis of the machine. Then we prop it up on little stacks of shims and pack in some grout, generally very wet grout around these shims. Unless I am very much mistaken, the grout either crumbles or shrinks away from the machine and leaves it sitting on the shims to be wracked out of shape as soon as the machine is put to work. We take care to get a good true bearing surface at all joints except that essential one at the top of the foundation.

About fifteen years ago, I found that such practice, which had been carried over from machinery to structural frames, was being abandoned. The Dominion Bridge Co., in Montreal, started about that time to have column bases poured a little high and dressed down by hand to exact

*"Concrete Operations in the Concrete Ship Program, *Proceedings*, V. 41.
†Port Arthur, Ontario.

elevation and the true level condition. Then they set their columns up, on small jobs right on the concrete, on important work using a sheet of lead under the steel to take up any minor variations. This method so appealed to me that I wrote it into a specification for a structural steel frame to be built in another city about five years later. What was my horror to find the steel subcontractor coming to me and begging to be allowed to use shims and grout. This was more remarkable as I had given the job of dressing the concrete to the general contractor. What he was afraid of, other than the unknown, is a mystery to me to this day, for I did not consider the point important enough to fight about and let him have his way.

In the years that followed, I have become more and more convinced that the method of dressing the top of the foundation is the proper method to follow, for it is similar to the method used at all other mating surfaces. It is just as easy, perhaps easier, to dress concrete as steel, and I'm all for the practice.

What holds us back? I think the answer is "fear". Men are afraid that, if some novelty in setting a machine is used, and that machine subsequently gives trouble for any reason at all, the manufacturer will blame the method of installation, for all the trouble. Well, he might try, but that doesn't say he would necessarily get away with it. Suppliers of materials and equipment will try it on with anybody they think it profitable to fool until the end of the world; it's only a part of the game and its possibility need never worry anyone who knows his business.

By CHARLES MACKLIN*

To my knowledge the only manufacturers who consistently recognize the necessity of incorporating in their designs a positive means of adjusting objects placed on concrete are the manufacturers of electrical floor junction boxes.

Setting base plates can be done by any of the five schemes shown in Fig. 1 and each may serve a particular need. However schemes 3, 4 and 5 are more accurate and economical. They do however require foresight and planning. When base plates are inaccurately set erection of structural steel is complicated and unnecessary stresses are introduced.

Schemes 1 and 2 require the use of four supports, one near each corner of the base plate. Schemes 3, 4 and 5 are better using only three points of support but four may be used.

The shrinkage of grout may be assumed to be about 1/100 of the thickness of the grout when the grout sets under no pressure and no packing is done after filling. This value may be appreciably reduced by

*Springfield, Ill.

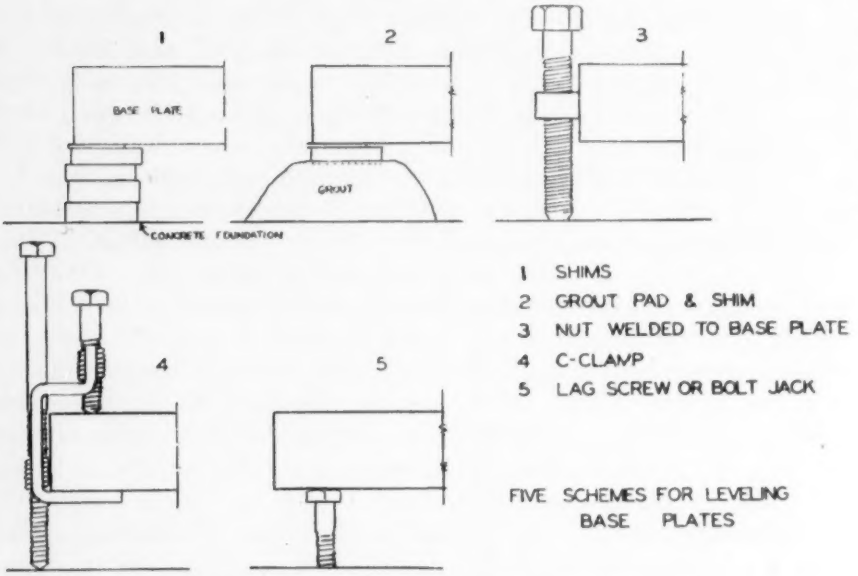

Fig. 1—Five schemes for leveling base plates

filling the grout higher than the bottom of the base plate and allowing settlement to take place for a half hour and then by exerting pressure on the forms and applying some hammer blows the excess water rises and spills over the form. This process may be repeated until the grout no longer responds to pressure. In applying pressure to the forms two methods can be used. One is by using a clamp and the other is by building a rectangle of wood around the form and applying pressure with jacks or wedges. In either case the form should be limber enough to bulge noticeably from the pressure of the grout and after the settling period this deflection can be reduced or eliminated.

Mr. Tyler recommends the procedure of introducing the grout through a pipe which of course does no harm. It is of little advantage unless the form is sealed to maintain a pressure over any appreciable area. The pressure gradient in grout may be very steep and any openings between the form and the plate make it possible to lose this pressure in a very few inches. When the grout completely fills the space between the foundation and the base plate, pressure exerted from any outside source does help to settle the grout, provided there is some place to which the excess will escape. The settlement can be speeded up by vibration. However, since this time element is present, unless pressure is exerted after some settlement has taken place, not much reduction in shrinkage will result from high initial pressures.

The article "Concrete Operations in the Concrete Ship Program" ACI JOURNAL, Jan. 1945 by Lewis H. Tuthill describes the procedure for patching ship hulls which is essentially the operation of patching between the foundation and the base plate.

When they receive the column loads base plates deflect and, coming in contact with the hardened grout, relieve the local pressures on any type of edge support. The possibility of a few thousandths of an inch settlement to make contact with the grout is of slight practical importance.

Since bearing pressures rarely exceed 1000 psi and 2 or 3 times this strength is readily obtained from wet mortars, the object of using a grout of any drier consistency which makes placing difficult seems to this writer absurd because the greater settlement shrinkage of the wetter concrete can be taken care of with pressure. The research of Giesecke (Proceedings ASTM part 2, 1920 page 219) indicates that rodded concrete increases in strength in spite of the water used. The grout is compacted and increased in strength whether worked by pressure on the forms after a time interval, or rodded.

Q—Now the further related problem: Shall the pipe sleeves that encase long holding-down bolts of a heavy machine be filled with grout when, after shimming up the machine in proper alignment, grout is forced between the machine base and the concrete foundation?

By W. ROWLAND*

A—The purpose of the sleeves is to allow enough movement of the top of the bolt to correct inaccuracies in setting the bolts.

After the machine base is set with the bolts fitting properly, the sleeves should be filled with grout for two reasons: 1) no more adjustment of the bolt location is required; 2) By resisting in sheer (as they should in conjunction with friction between base and foundation) bolts can prevent any horizontal movement of the machine only when they are grouted in right up to the underside of the machine base.

By L. C. HANSON*

A—On a five-stage horizontal, steam-driven compressor of which the pipe sleeves had been filled with grout, some of the anchor bolts broke off near the top of the grout. When in operation the machine (moving 0.005 in. back and forth) bent the projecting part of the bolts once back and forth with each revolution. Because the projection measured only 6 in. the bending moment was so great that the bolts broke from fatigue. Had the pipe sleeves not been filled with grout

*The Solvay Process Co., Hopewell, Va.

the projecting part would have been approximately sixteen times as long and consequently for the same deflection the stress would have been 1/256th as great (sixteen to the second power) and therefore safe. My conclusion is that sleeves should not be filled with grout.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Concrete overlays for airfield runways

Engineering News-Record, V. 133, No. 24 (Dec. 14, 1944), pp. 80-82 Reviewed by S. J. CHAMBERLIN

Concrete runways were strengthened to carry a 37,000-lb. wheel load by a 6-in. overlay of reinforced concrete. The old concrete was brushed clean, primed with hot tar and coated with a one-half inch layer of asphaltic concrete to act as a cushion and to prevent bonding with the new concrete. Portable pierced-plank steel runways served as emergency extensions during the remodeling to provide continuous operation of the field. New concrete runways have a thickness of 10:7:7:10 in. on a 4-in., dry-bound macadam base.

Prestressing bands on a concrete tank

John J. CROWLEY, *Engineering News-Record*, V. 134, No. 18 (May 3, 1945), pp. 116-118
Reviewed by S. J. CHAMBERLIN

In building a 300,000-gal. prestressed concrete water tank, well known methods were used until the time came to get the right tension in the steel bands. After field and laboratory studies were made the extensometer method was abandoned and a shear wrench method developed. The shear wrench method consisted in tightening the nuts on the ends of the bands until the wrench handle sheared a pin of predetermined strength. Nuts and washers were coated with flake graphite mixed with oil. Laboratory calibration of the shear pins indicated 1/16-in. bronze Oxweld brazing rod suitable for the preliminary prestressing of 16000 psi in the ¾-in. square bars and 1/16-in. steel Oxweld welding rod for the final stress of 31,500 psi. It was concluded that refinement in stress control (within 10 per cent) should not be expected.

Rigid-frame concrete span requires no falsework

JACOB FELD, *Engineering News-Record*, V. 134, No. 18 (May 3, 1945), pp. 105-106
Reviewed by S. J. CHAMBERLIN

Structural steel frames are first designed as rigid frames to sustain the weight of forms and fresh concrete. When the concrete has hardened the structure becomes a reinforced frame, the structural angles acting as reinforcement in both faces. Three costly items can be eliminated: 1) supporting arch forms on falsework; 2) bending and placing long and heavy bars; and 3) separating and holding in proper position the intrados and extrados reinforcement layers. Structural steel ribs are spaced 24 in. on centers and are formed of double 4 x 4-in. angles in each chord. Diagonals are single angles and the struts are Z bars. The proposed structure will have a clear span of 80 ft. 6 in. and a width of 100 ft.

Prestressed concrete design overseas

HENRI MARCUS, *Engineering News-Record*, V. 134, No. 14 (Apr. 5, 1945), pp. 97-100

Reviewed by S. J. CHAMBERLIN

European engineers have made considerable progress in the application to concrete of artifically induced stress. Short descriptions of several examples of prestressing are given (particularly the work of Edouard Freyssinet) including long span beams, slender bridges, increasing the heights of dams and precast caissons. Successful application involves adjusting the prestressing forces to the strain produced by natural forces, the use of adequate tubes for easy manipulation of the reinforcing cables and special grouts for their protection, and specially built hydraulic jacks for rapid and intensive stressing and anchoring of the wires.

Railroad tunnel roof rebuilt from shaft

Engineering News-Record, V. 134, No. 8 (Feb. 22, 1945), pp. 70-74

Reviewed by S. J. CHAMBERLIN

Faced with the problem of replacing an old brick roof with a concrete arch without interrupting traffic, the contractor attacked the job from the top by sinking a vertical shaft and driving a crown drift above the damaged section. The area was then enlarged and shored to the old lining for a length of 10 to 15 ft. Haunch concrete was then placed and 7-segment timbering placed to which the roof supports were transferred. Center sections of the brick arch were removed and all areas above the lagging were back packed with old brick. Forms for the concrete arch were supported by the old timbering that had previously held the damaged brick section. The concrete was then pumped into place for the arch followed by concreting above the arch. Arch forms and the old timber support system will remain in place until the work is completed.

Engineering for dams

WILLIAM P. CREAGER, JOEL D. JUSTIN and JULIAN HINDS. 3 vols. John Wiley & Sons, New York; Chapman & Hill, London, 1945. 929 pp., illus., diagrams, tables, charts, cloth, 9¼ x 6 in., $16.50.

Reviewed by R. F. BLANKS

A very useful compilation of data and information on the investigation, design and construction of all types of dams, and also on related hydraulic problems, skillfully condensed to usable form. Volume I covers general investigation and design problems including geology, foundations, selection of type, model studies and hydrology. Concrete engineers will find particular interest in Volume II, which deals, in detail, with the design of non-overflow and spillway types of solid gravity dams, arch dams and buttressed, multiple arch and other types of concrete dams. Chapter 15 is devoted exclusively to concrete and concrete materials for concrete dams. Volume III treats the investigation, design and construction of earth, rockfill, steel and timber dams. Soils tests, construction control and related subjects are given particularly prominent emphasis. The authors were assisted by several eminent authorities on dam and hydraulic engineering.

Use of all-in aggregates in concrete

A. R. COLLINS, *Concrete and Constructional Engineering*, V. 40, (Jan. 1945), pp. 15-19.

Reviewed by GLENN MURPHY

The author describes expedient measures for producing concrete of reasonable quality in emergency construction where the available aggregate cannot be washed or graded. Studies indicate that the principal cause of deterioration is silt, clay, or other fine material which increases the water requirement. The resultant high water-cement ratio decreases the strength and durability of the concrete. The need for holding the water-cement ratio constant by increasing the cement content at the same time the water

content is increased to maintain workability is pointed out. It is also noted that the resultant rich concretes may have a tendency to shrink and crack. Standard field techniques for making sieve analysis and for detecting the presence of silt and organic impurities are described. The author outlines a trial batch method for field design of the satisfactory mixture. The paper includes a table of gradings for assistance in combining aggregates to improve the grading.

Reconditioning concrete floors to carry heavy warehouse traffic

Engineering News-Record, V. 134, No. 4 (Jan. 25, 1945), pp. 86-89 Reviewed by S. J. CHAMBERLIN

The article is extracted from "Toppings and treatments for the correction of concrete floor deficiencies" compiled by the Repairs and Utilities Branch of the Office of the Chief of Engineers of the Army. The first step in resurfacing with concrete is to cut out the defective surface to a minimum depth of 2 in. with vertical and straight edges. After vigorous cleaning it is recommended that the prepared surface be thoroughly saturated with water, later removed with a mop so the top will be in a surface-dry condition. A cement-paste slush coat should then be brushed in and the excess removed. Recommended proportions for the inlay concrete are 1:1 1/3:2 with a water-cement ratio of 4 to 4½ gal. per bag. Careful finishing and curing are essential. Asphalt mastic for use as an overlay in resurfacing should be a combination of portland cement, asphalt emulsion, water and sand, with a ½-in. minimum thickness and feathered at the free edges. Sodium silicate solution and magnesium fluosilicate solution are recommended to retard "dusting" of floors.

Cement-treated road base built in Arizona

BERNARD TOUHEY, *Engineering News-Record*, V. 134, No. 2 (Jan. 11, 1945), pp. 108-110
Reviewed by S. J. CHAMBERLIN

The first step in the construction of the mixed-in-place base was spreading of the sand and gravel aggregate the full width of the roadway to a depth sufficient to assure a compacted thickness of 6 in. Cement was spotted on top for an approximate mixture of three sacks per cubic yard of compacted material. After motor patrols windrowed the material the dry aggregate and cement were mixed by one of the traveling plants. Water was then added by the second traveling plant and the third plant completed the wet mixing. The traveling mixing plants were pugmills swung under grader frames. Material was picked up in the pugmill and then forced back into the revolving spools to which the mixing paddles were welded. Mixed material was forced out the rear of the mixing box in a uniform flat-topped windrow. A blade grader followed immediately behind the third traveling plant. The mixture was rolled with both steel and pneumatic rollers and then sprayed with emulsified asphalt for a curing seal. Subsequently, a 2-in. plant-mix bituminous seal coat was added. Twenty-eight-day compressive strengths of the material averaged 2,037 psi.

Concrete construction in cold weather and during frost

Revision of paragraph 30 of obligatory specification for mortar and concrete (AMB) of the German State Railways (Deutsche Reichsbahn): *Asph. u. Teer*, 1944, 44 (¼), 27-8. Road Abstracts, Vol. XII, No. 1, January 2, 1945. HIGHWAY RESEARCH ABSTRACTS

The requirements for concrete construction in winter are given in greater detail than in the published account of the earlier revision of paragraph 30 of the specification for mortar and concrete. The following additional points may be mentioned: Rapid-setting cements and strongly exothermic cements are most suitable for winter construction; German aluminous cements are almost unaffected by low temperatures. In frosty weather the proportion of cement may be increased. The proportion of mixing

water shall be kept as low as possible by choosing a suitable grading for the aggregates. The concrete should ordinarily be "earth damp" (a somewhat wetter mix is necessary if aluminous cement is used) and should preferably be vibrated. Before being charged, the mixer shall be washed out with hot water. If the aggregate and water are preheated the cement shall not be added until it has been ascertained that the temperature of the mixture does not exceed 40 C. (122 F). The temperature of the concrete in the mixer shall not be below 20 C. (68 F.). The only admixture permitted is calcium chloride (90 to 93 per cent pure), to be used in aqueous solution, and then only at temperatures down to -15 C. (+5 F.). For earth-damp concrete the proportion of calcium chloride should be 1 per cent of the cement used; 2 per cent may be used in wetter mixes.

Expansion joints unnecessary in pavements

Engineering News-Record, V. 134, No. 12 (Mar. 22, 1945), pp. 100-103 Reviewed by S. J. CHAMBERLIN

After three years of observations on two experimental paving projects the Oregon State Highway Department has decided that expansion joints are not needed in Western Oregon provided dummy (contraction) joints are properly spaced. On one project expansion joints of several types were spaced from a minimum of 30 ft. to a maximum of 495 ft., with dummy joints placed every 15 ft. The other test section was placed in two 1,320-ft. sections without expansion joints, but with dummy joints again spaced at 15 ft. The opening and closing of both types of joints were determined with a strain gage, and longitudinal displacements were measured from a transit line across the pavement. The indications were that a relatively rapid closure of expansion joints may be expected during the first season after construction, since most of the expansion between joints must be taken up by the slabs sliding along the subgrade. During the following winter contraction of the concrete opens the dummy joints, with expansion during the second and subsequent summers taken up by closing of the dummy joints. Although only about half of the dummy joints opened during the first winter all opened during the second winter. Vertical displacement of adjacent pavement sections under a 20,000-lb. axle load were measured with a displacement meter to determine the load-transfer property of the expansion joints. Results indicated that a joint with a concrete sill could be substituted for the usual doweled joint with an increase in rigidity.

Observations reveal deformations of steel reinforcement

S. SERGEV, *Civil Engineering*, V. 15, No. 2 (Feb., 1945), p. 92. Reviewed by J. R. SHANK

Strain readings have been taken from time to time on the steel of reinforced concrete columns and beams of Biology Hall of the University of Washington. The basement columns, $A = 325$ sq. in., $A_s = 6.17$ sq. in., of a 1:1:2 mix under a load of 160,000 lb. showed an average unit stress on the steel at 10 years to be 30,000 psi. First story columns, same cross-section of 1:2:4 concrete showed 22,000 psi under a load of 136,000 lb. Readings on opposite faces in both cases showed deviations from the average of 20 per cent in the basement columns and about 10 percent in the first story columns showing the effects of deformation stresses. Though the author speaks of this increase in unit stress as being due to shrinkage it is obvious that it is due to a combination of shrinkage and plastic flow. Previous data on plastic flow for similar concretes show an increase to 23,000 psi may be expected for the basement columns, leaving 7,000 psi of the increase due to shrinkage, which may be expected.

Strain readings on the tension steel at mid-span points of beams showed a maximum tension of about 10,000 psi to have occurred at 30 to 40 days, after which a slow reduction was had, leaving an observed stress of only 2,500 psi at 10 years. This is one-fourth what calculations would indicate for simple beam action.

Bonneville Dam concrete after six years

C. C. GALBRAITH and R. R. CLARK, *Engineering News-Record*, V. 134, No. 10 (Mar. 8, 1945), pp. 121-124
Reviewed by S. J. CHAMBERLIN

Considering the behavior of the mass concrete to be fully as important as that of the surfaces exposed to erosive action of water, the selection of a modified portland cement ground with 25 per cent pozzolanic material was based on the decision to build the blocks rapidly without vertical joints parallel to the axis. The present condition of the concrete surfaces of the superstructure is approximately the same as at the end of construction. The spillway dam discharges large flows for long periods and severe erosion has occured at local zones at the bases of all piers just downstream from the gate slots and on the curved overflow sections adjacent to the piers. It has been concluded after seasonal repairs with different brands of cements and admixtures that concrete is not suitable for surfacing these localized areas, and they have therefore been covered with ½-in. steel plate. Some of the steel plates have been torn away, but from the experience gained to date it will be possible to maintain these surfaces satisfactorily without excessive cost. Another localized zone of erosion is the downstream baffle deck. The erosion is localized on the vertical face of the triangular baffles and on the horizontal deck at the downstream corners of the upstream row of baffles. At one spot the deck has eroded to a depth of 32 in., with depth ranges to 18 in. in spots at all bays. Repairs with standard cement and a richer mixture seem to withstand the action of the high velocity water somewhat better than the concrete made with pozzolanic cement. The deck is about 20 ft. thick and it is thought that it will not require anything more than the renewal of the surface at intervals of 15 to 20 years (a more favorable condition than was anticipated at time of construction). The interior of the mass concrete appears to be in a satisfactory condition.

High-strength reinforced concrete column developed

GEORGE A. MANEY, *Civil Engineering*, V. 14, No. 12, (Dec., 1944), pp. 496-498.
Reviewed by J. R. SHANK

Reinforced concrete column specimens have been developed at the Northwestern Technological Institute capable of withstanding as much as 57,000 psi in compression. A number of important properties of concrete and steel and combinations of these were employed in this development among which were the high strength that may be developed in portland cement products when vibration cast under high pressure using very low water contents and the high value of spiral column reinforcement when under tension. Water contents as low as 1 gal. per sack of cement and pressures up to 18,200 psi were used.

High strength steel wire was wrapped on a tubular steel section into which very dry concrete was placed and vibrated strenuously under high pressure for a minute, after which the vibration was stopped and the pressure was removed. It was found that the concrete was strong enough to maintain the spiral steel in initial tension. A neat cement specimen, 1 gal. of water per sack of cement, 11 percent of high-strength spiral wire and 3.3 per cent of tubular liner by volume, under a forming pressure of 11,8000 psi developed 47,000 psi strength. The stress-strain curve was straight up to 12,000 psi indicating a modulus of elasticity of 9,400,000 psi. Another specimen 8.3 in diameter, 1:0.5:0.715 mix by weight, 2½ gal. of water per sack, 19 percent spiral wire and 12 percent liner by volume, formed under a pressure of 13,500 psi reached 55,000 psi and showed an initial E value of 8,500,000 psi.

Further research is necessary to find the best types and proportion ranges for spiral wire and tubular steel, as well as forming pressures and concrete mixtures. It is apparent now that units can be manufactured which will have weights comparable to struc-

tural aluminum and have higher moduli of elasticities at costs as low as one-third that of structural steel.

Deteriaration of concrete dams due to alkali-aggregate reaction

R. F. BLANKS and H. S. MEISSNER. *Am. Soc. C. E. Proc.* V. 71, No. 1 (JAN. 1945) p. 3-18
Reviewed by H. J. GILKEY

Constitutes a progress report based on close continued observation of concrete dams and other structures and on continuing laboratory studies. Some structures develop signs of surface damage rapidly (within two or three years) and the rate of damage may soon decrease greatly. Other structures may develop initial evidence of damage much more slowly but the progress of deterioration may be continued over a long period. The characteristic large surface cracks do not extend far inward but are the result of interior swelling as demonstrated by decreased core strengths and greatly decreased concrete stiffness (lowered modulus of elasticity) of material from the interior. The swelling has been further demonstrated by chord measurements on arch dams and by measured up-stream movement of the arch crown, misalignment of machinery and by the rising of top surfaces in amounts proportional to depth of concrete. The laboratory mortar bar accelerated tests give valuable indications but the correlation with different actual structures is far from perfect. Most "good" aggregates are probably reactive to the alkalies of the cement in some degree but thus far no trouble has been encountered with cement for which the $Na_2O + K_2O$ is less than 0.60 percent. One of the difficulties in studying any but the most recent of structures is the absence of knowledge regarding the alkali contents of the cements that were used, since such determinations were not a part of normal tests. Representative samples of the aggregates can usually be secured, however. In spite of the many things not yet known regarding the problem the evidence thus far indicates that the use of a low alkali cement (under 0.60 per cent) is a desirable, and to date the only reliable, precaution against possible damage. This may add anything from nothing to $0.20 per barrel to the cost of the cement but the absence of the alkali produces other beneficial effects in the concrete such as reducing the C_3A (tri calcium aluminate) and the C_4AF (tetracalcium alumino-ferrite) with corresponding increases in the desirable constituents C_3S (tricalcium silicate) and C_2S (dicalcium silicate). There is some evidence that pozzolanic cements may safely be used with reactive aggregates and that the addition of pumicite to the cement is helpful. A list of eight selected references ties this paper to the preceding published observations on this vital subject.

South American building codes

ARTHUR J. BOASE, *Engineering News-Record*, V. 134, No. 16 (Apr. 19, 1945), pp. 68-77
Reviewed by S. J. CHAMBERLIN

The author points out the main similarities and great dissimilarities between the building code of Brazil and the American codes (especially ACI 318-41). The engineers of Brazil have organized the Brazilian Association of Engineering Codes and this group has issued the six codes that refer to the calculation and erection of structures. Unit stresses and most other details as set up in one code apply to all reinforced concrete construction and these codes are used by everyone engaged in engineering work. The Brazilian code is direct and simple leaving much to the engineer. Specified minimum live loads are tabulated in the article and compared with those used in New York City. Provision is made in the framing of tall buildings for less than 100 per cent loading on lower floors. Wind pressure is assumed as a function of the height and is designated as q, while the pressure as a function of the type and form of construction is designated as c. Values of q and c are listed. The strength of concrete used is much lower than

that used in this country. Reinforcing steel for the most part is plain and corresponds to our structural or intermediate grade. Permissible unit stresses are tabulated and compared with the ACI code as well as the German and French codes. Steel stresses in flexure of 21,300 psi and 25,000 psi in structural grade and intermediate grade respectively are permitted. Steel is about 10 cents a pound. One provision on bond stresses states that where bars or bundles of bars with a diameter of less than one inch are used in flexural member the bond stress need not be calculated, but all bars over ¼-in. in diameter must be hooked. Another provision states that where there are bent bars calculated to resist all diagonal tension stresses, twice the perimeter of the unbent bars may be assumed in bond calculations. Steel protection thicknesses are listed and compared; the protection being much thinner in Brazil. The minimum width of beams and girders is set at 3.2 in. and the designers very often stay very close to this minimum. Beams and girders, one-way slab design, joist floor construction, two-way slab construction, mushroom flat slab design and column design procedures are discussed and compared, with one of the basic differences between the Brazilian code and the ACI code being the handling of continuity. The Brazilian code permits design on an ultimate design basis, that is, it is permitted to design flexural members as a function of the rupture load.

Extend the life of concrete pavements—hints to maintenance men on patching with portland cement

A. A. ANDERSON, *Civil Engineering*, V. 15, No. 3, (Mar. 1945), pp. 131-134. Reviewed by J. R. SHANK

The determination of areas to be patched involves consideration of the condition of the existing pavement and the shape and dimensions of the patch. Patches are divided into five classifications:

1) full width patch; 2) half-width or single-lane patch; 3) exterior edge patch; 4) interior edge patch; 5) interior or plug patch.

The minimum length of a patch on one side of an expansion joint is 6 ft. and at a contraction crack, 8 ft. The expansion joint may be moved to the end of the patch where the area to be patched lies on both sides of an expansion joint if the patch is full-width on both sides. Except for full-width patches, expansion joints should be reproduced in their original positions. Rectangular, or diamond-shaped interior or plug patches, should be not less than 4 ft. nor more than 6 ft. laterally from the center line and not less than 4 ft. long. For diamond-shaped patches at some distance from joints, the angle between the sides of the patch and the center joint should not be less than 30 or more than 60 degrees. At transverse expansion joints this angle should not be less than 30 or more than 45 degrees.

It is recommended that patches be of uniform thickness and that dowels and tie-bars be eliminated. The thickness of the patch should be of the same thickness as the existing slab for uniform thickness slabs, except at unprotected corners where it should be 10 per cent thicker, to 1.3 times the center thickness at unprotected corners for thickened-edge slabs, or, at other places, 1.2 times the center thickness.

Edges of patches should be straight and vertical. The sides of rectangular patches are usually parallel to and at right angles to the center line. Edges may be the original crack where the angle is not less than 30 degrees with the center line.

Concretes should be equal in specifications to, or better than the original concrete and high early strength materials and methods are usually advisable. A modulus of rupture of 500 psi is recommended as the strength at which to open patches to traffic. The concrete should be proportioned and mixed according to best practice and tamped nto place against dampened edges of old concrete. After delaying as long as possible,

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while still permitting the surface to be finished, the concrete should be tamped a second time. Best appearance results if patches are finished with a surface texture as nearly as possible like that of the original pavement.

Sawdust-cement

BUILDING RESEARCH STATION: R.I.B.A.J., 1943, 50 (11), 259-63.

BUILDING SCIENCE ABSTRACTS

Data obtained and conclusions drawn from recent investigations on sawdust-cement conducted at the BUILDING RESEARCH STATION, Garston, are here summarized for the benefit of manufacturers and users. The shrinkage and expansion of sawdust-cement products are discussed and measures for reducing these movements as much as possible are detailed. For precast units the cement/sawdust ratio should be as high as possible, i.e., 1: 2 or 1 : 3 mixes, and the products carefully matured for 4 to 6 weeks, the last stages being done under cover in the open. Residual movements when the products have been built into a structure, i.e., shelving, flooring, etc., may be further reduced by waterproofing with a paint or by a tar or bitumen. Another alternative is to design the method of fixing so as to permit some freedom of movement (1) by using tongued and grooved plank-shaped units with dry joints, (2) by holding roofing units to rafters with simple spring clips and covering these specially shaped units with clay or concrete tiles which can be nailed down, (3) by nailing or bolting flat slabs on roofs leaving the holes sufficiently large for movement and then covering the top surface with roofing felt. Uniformity in weight, strength and setting of sawdust cement may be ensured by (*a*) addition of hydrated lime to the cement-sawdust mix in the proportion of $\frac{1}{6}$ to $\frac{1}{3}$ volume per 1 volume of cement used; this method, which improves setting and strength development, is applicable to batches of the normal mixed wood sawdusts containing a minimum of larch, or (*b*) pre-treatment: a method equally effective with all sawdusts is outlined. Some points to be observed in the manufacture of pre-cast sawdust-cement products are briefly noted. The use for floors of small pre-cast blocks having a sawdust-cement base has proved satisfactory and it is suggested that a jointless flooring composition of the sawdust-cement type might be an efficient substitute for timber flooring or jointless surfacings which are now unavailable or in very short supply. Plain sawdust-cement mixtures, however, tend to lift or crack, so that to minimize this defect it is advisable to lay the mix immediately on fresh concrete or to use cement-sand-sawdust mixes ranging from 1 : $1\frac{1}{2}$: $1\frac{1}{2}$ to 1 : 1 : 2 whereby drying shrinkage is considerably reduced. Such mixes, too, may also be laid on old concrete if suitable precautions, here outlined, are taken. Some of the existing and possible uses of sawdust-cement are summarized and finally consideration is given to the question of appropriate performance tests for sawdust-cement products to meet specification requirements. Quantitative data regarding the physical properties and strengths of various cement-sawdust mixes are presented in four tables.

Actual and estimated impact resistance of some reinforced-concrete units failing in bending

L. G. SIMMS. *Journal of the Institution of Civil Engineers* (London).

HIGHWAY RESEARCH ABSTRACTS

What little attention has been given to the behavior of reinforced concrete units under impact has normally been confined to analytical considerations applicable to the point where either the yield of the steel or the crushing of the concrete is reached. While the best way to determine the resistance to impact of reinforced concrete is, and probably always will be, practical trial, there is some evidence that an approximate analysis of transference of energy may, in certain simple cases, lead to the order of resistance to be expected.

It is known that the criterion of resistance for loads acting over a short period is capacity to absorb energy; but since it has been usual practice to discontinue static tests on reinforced-concrete units when the maximum load is reached, few data are available as to energy-absorpton even under static loading.

In this report the beams tested were 7 ft. long, 4 in. wide, and 8 in. deep and the corresponding dimensions of the slabs were 6 ft., 20 in., and 6 in. respectively. In the beams, stirrups were provided to ensure that the failure was in bending without earlier shear failure. In general the longitudinal reinforcement consisted of mild steel bar, the quantity of steel ranging from 0.1 per cent to 3 per cent. In a few tests a high-strength steel was employed. The reinforcement in the slabs consisted of three types of cold-worked steel, ribbed mesh sheet steel, and three steels sold as mild steel.

The average crushing (cube) strength of the concrete employed in the beams was 3,000 psi, and of that in the slabs 3,800 psi.

Under load all specimens were simply supported, with restraint to the ends reduced to the minimum. The load, either static or impact, was transmitted to the concrete through a steel bearing-plate located at the center of spans of 6 ft. and 5 ft. for the beams and slabs respectively.

In the impact tests the deflections were obtained by a stylus fixed to the specimen and travelling with slight pressure over waxed paper fixed to a plate. The stylus, in removing a line of wax coating, revealed the red surface of the paper behind.

The impact testing apparatus was simply a frame with guides for a weight dropped from various heights. It is realized that with this type of apparatus some of the available energy is lost before impact, owing to friction between the weight and the guides, but it is considered that the efficiency of the machine was well above 90 per cent. In all cases it was arranged that the weight of the falling mass was equal to that of the unit under test. The height of the frame was such that the limiting velocity of the falling mass on impact was 30 ft. per sec.

It is important to note that in these experiments the conditions of test were as follows: 1) all specimens were designed to fail in bending; 2) the impact was characterized by local deformation; 3) the weight of the falling mass was equal to that of the specimen; 4) the striking velocity of the falling mass varied over a range from 8 ft. per sec. to 28 ft. per sec.

The tests showed that, in general, the form of damage to some simply-reinforced concrete units was of the same type under impact as under static loading. In such cases reasonable estimations of damage due to impact were obtained from an equation in which a reduction factor had been derived from considerations of an elastic material.

The tests have shown the limits of deformation in bending of some simple reinforced-concrete beams and slabs occurring after the steel had yielded or after the concrete had crushed.

In general, the form of damage to these units was roughly the same under either static loading or impact, and in such cases, the damage to beams and slabs due to impact was reasonably predicted from considerations of the energy absorbed under static loading, used in conjunction with a simple energy-equation.

Where the impact strength of a reinforced-concrete slab was considerably overestimated, the poor strength was shown to be associated with the low impact strength of the steel employed as mild-steel reinforcement. The very poor performance of this steel under impact was not indicated by the usual routine mechanical acceptance tests, and it is considered that when the tension decides the capacity of reinforced-concrete units to absorb energy, the impact strength of the steel should be tested specially.

Destruction of concrete by frost

ARTHUR RICHARD COLLINS. *Journal of the Institution of Civil Engineers*, No. 1 (Nov. 1944), (London)
HIGHWAY RESEARCH ABSTRACTS

Under wartime conditions, with the necessity for speed of construction and shortages of suitable labor and materials, it has not always been found possible to apply normal peacetime standards, and concrete of very low quality has sometimes been produced.

In view of the serious nature of the failures, an investigation was begun at the Road Research Laboratory on the general subject of the frost-resistance of concrete and a study was made of the mechanism by which damage is caused.

By means of a dilatometer the total volume of the water and the material in which it was absorbed was measured over a range of temperatures. With a reduction in temperature the normal contraction in volume occurred, but when any part of the water froze the volume-temperature relationship was interrupted by the increase in volume of the water. By plotting the volume against temperature the freezing-point of the water in the voids was determined. This technique has been applied to concrete and in an illustrated case the period of ice-formation extended from about − 3 to − 4 C. in the case illustrated.

An important factor in the formation of ice in porous materials is that ice crystals in contact with water grow parallel to the direction in which the heat is flowing, and not necessarily in the direction of least resistance. The crystals will continue to grow so long as a supply of water is available or until the pressure attains a point at which freezing is inhibited. This pressure which increases as the temperature falls, reaches a maximum of about 30,000 psi at — 22 C. and is exerted in a direction parallel to the axes of the ice crystals; that is, normal to the cold surface.

The mechanism of the failure of concrete during freezing can be vizualized as follows: Cooling begins at the exposed surface and extends slowly inwards. When any layer below the surface reaches a sufficiently low temperature, the water in the largest pores begins to freeze and the latent heat given up by it tends to maintain constant temperature at the point of ice-formation. The ice crystals so formed are in contact with unfrozen water in the surrounding smaller pores and, by drawing water from them, the crystals continue to grow.

As already stated, the force exerted by the ice will be perpendicular to the cold surface, and, if the concrete is of low strength, a plane of weakness parallel to the cold surface will tend to form at the level at which the ice is forming. The water drawn in by the growing crystals of ice will come first from the largest of the unfrozen pores. As these become emptied the supply will be restricted and the rate of growth of the ice will be checked. The evolution of latent heat will not then be sufficient to maintain the temperature constant at the point of ice-formation and the temperature will begin to fall once more. As there is then little or no water in the largest pores in the concrete immediately below the first ice layer, freezing will not begin again until the cold front has penetrated some distance, that is, until either the temperature has dropped sufficiently to freeze the pores that do contain water or a level is reached where the larger pores are not affected by the ice forming above them. The result of this process is that the concrete will contain a series of planes of weakness parallel to the surface of cooling. During subsequent cycles of freezing the ice will again tend to form at the same levels as before, because the pores there have been distended by the previous ice, and the freezing-point of the water in them will be higher than in the surrounding concrete.

The damage to the concrete is considered to be caused, not so much by the actual initial increase of volume of the water in the pores on freezing, as by the growth of the crystals afterwards and the consequent segregation and concentration of the ice into the layers.

Laboratory evidence demonstrating the difference between wet and dry freezing is given in Table 1, which shows the steady reduction in crushing strength of concrete frozen wet while that frozen dry continued to increase in strength though naturally at a lower rate than when cured at normal temperature.

TABLE 1—EFFECT OF WET AND DRY FREEZING ON THE CRUSHING STRENGTH OF CONCRETES

Number of cycles of freezing and thawing	Crushing strenth expressed as a percentage of the strength at 7 days		
	Frozen wet	Frozen dry	Cured at normal temperatures for same period
0	100	100	100
10	141	165	189
20	137	189	240
30	119	201	263
40	99	211	304
50	63	220	332
60	0	228	354

All specimens were cured at normal temperature for 7 days before freezing.

Conclusions. The main points raised in this paper may be summarized as follows: From the evidence available it appears that frost damage in concrete and other porous materials is due to the segregation of ice into layers rather than to the simple expansion of the frozen water in the pores. The mechanism is, in fact, similar to that occurring in soils during frost heaving.

The conditions favoring severe frost damage are concrete of high porosity and low strength, the presence of water within or in contact with the concrete, and cooling to a temperature several degrees below freezing-point over an appreciably long period.

The frost-resistance of concrete, like many of its other properties, is determined largely by the water/cement ratio, and the best way of obtaining good durability is by exercising proper control over the mixing operations. A nominal water/cement ratio of 0.70 by weight is suggested as a maximum for exposed concrete in emergency construction, but lower values than this would normally be required in peacetime.

An improvement in frost-resistance can be obtained, at the expense of a possible reduction in crushing strength, by the use of certain admixtures as tallow, resins and organic foaming agents.

Once concrete of low quality has been placed, little can be done to protect it from damage except by restricting access to moisture and protecting it from extremes of temperature.

ACI NEWS LETTER

Vol. 16 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1945

ACI Book of Standards

In conformity with a decision of the Institute's Board of Direction, to republish ACI Standards in their adopted form (including convention revisions and editorial corrections), the *Proceedings* pages of this JOURNAL are chiefly devoted to the Institute's Standards adopted since the inauguration of the procedures under the Standards Committee in 1937. They have all been and will be available in separate prints and as soon as possible after the appearance of this June number they will become available for sale also in a single book publication. There will be re-publication of such books of Standards as rapidly as the appearance of new standards justifies.

It is well to call special attention to the fact that Standards adopted by convention action subject to revision and editorial correction, and subsequently ratified as revised by letter ballot of the Institute membership appear now in their corrected form.

New Members

The Board of Direction approved 39 applications for Membership (34 Individual, 4 Corporation, 1 Student) received in March and April as follows:

Bross, W. A., U. S. Engineer's Laboratory, University of Wash. Campus, Seattle 5, Wash.

Burns, Bennett W., P. O. Box 591, Baytown, Texas

Coff, Leo, 198 Broadway, New York 7, N. Y.

Colegio Nacional de Ingenieros Civiles de Cuba, Manzana de Gomez, Havana, Cuba

Foss, Frederic D., 256 Whaley St., Freeport L. I., N. Y.

Gilbert, J. R., 6503 Florida St., Chevy Chase, Md.

Hawley, A. J., 236 Harborview, Bridgeport, Conn.

Kavanagh, Thomas C., 2005 Coyle St., Brooklyn 29, N. Y.

Keil, J. P. G., American Viscose Corp., Marcus Hook, Penna.

Kensley, Phillip R., U. S. Engineer Laboratory, University of Washington Campus, Seattle 5, Wash.

Kuszewski, Zygmunt, 281 O'Connor St. Apt. No. 6, Ottawa, Ont., Canada.

Lashbrook, C. A., Okla. Testing Laboratories, P. O. Box 1982, Oklahoma City, Okla.

Lemke, Kenneth F., Box 183, Shawano, Wis.

Lyttle, James B., 635 East 211th St., Bronx 67, N. Y.

McCammon, G. A., Division Chief Engineer, Creole Petroleum Corp., Apartado 172, Maracaibo, Venezuela

McCoy, H. W., 303 Gary Ave., Wheaton, Ill.

McMillen, Dale S., 1621 Oakview Ave., Berkeley, Calif.

Maloney, Charles P., 3112 - K St., N. W., Washington 7, D. C.

Paulmier, C. M., c/o Oregon Shipbldg Corp., Adm. Bldg., Portland, Ore.

Pelarz, Enrique A., Ministerio de Agricultura, Havana, Cuba

Pritchard, E. O., Inversnaid Upton Wirral, Cheshire, England

Prokop, Charles J., 133-36 Dennis Ave., Springfield Gardens, Long Island, N. Y.

Quraishy, Saleh, Head, Dept. of Civil Engr., Muslim University, Aligarh, India

Robertson, C. R., 627 Arsan Ave., Baltimore 25, Md.

Rydland, Anton N., 515 Union Bldg., c/o T. V. A., Knoxville, Tenn.

Simmons, Allen, P. O. Box 185, Bethany, West Va.

Societe Egyptienne de Tuyaux, Poteaux et Produits en Ciment Arme, 15 Rue Madabegh, B. P No. 111, Cairo, Egypt (Henry Barcilon)

Sorhegui, Agustin, Metropolitana 238, Havana, Cuba

Sproule, F. R., Wyandotte Chemicals Corp., Wyandotte, Mich.

Stearns, Edward W., 117 Meadowbrook Pl., South Orange, N. J.

Stephenson, Henson K., Texas Engineering Experiment Station, A & M College of Texas, College Station, Texas

Sundt, M. Eugene, 343 North Hermosa, Albuquerque, New Mexico

Tinkler, J. W., 16 Gould Street, Toronto, Ontario

Vaughan, Albert P., 5023 West End Ave., Chicago 44, Ill.

Vilaret, Manuel, c/o Sinclair Oil Co. of Cuba, Banco Canada, Aguiar No. 367, Havana, Cuba

Wilson, Merle D., 1066 Berwin St., Akron 10, Ohio

Wellington Harbour Board, P. O. Box 938, Wellington, New Zealand

Warzyn, W. W., 7318 Church Ave., Ben Avon, Pittsburgh 2, Pa.

Wayne County Road Comm., Board of. 3800 Barlum Tower, Detroit 26, Mich, (Paul Holland)

Who's Who

R. L. Bertin, Joseph DiStasio and Maurice P. VanBuren

are all members of the Institute; are all pretty well known to the readers of this JOURNAL as their present paper, the only paper presented in this issue of the JOURNAL, is not their first contribution.

Mr. Bertin, Chief Engineer of White Construction Company since 1917, has been a member of the Institute since 1921 and a record of some of his major activities in the Institute will be found in the 1944 Directory under his name, page 38. He has served as a member of the Board of Direction; has been active in committee work; written several papers.

Mr. Di Stasio, member of the Institute since 1920, Joseph Di Stasio and Co., Consulting Engineers, New York, has served as a member of the Institute's Advisory Committee and has twice contributed papers to these *Proceedings;* once before on the subject of the present contribution.

Mr. Van Buren has been a member of the Institute since 1926; collaborated with Joseph Di Stasio in presenting "Slabs Supported on Four Sides" to Volume 32, p. 350, ACI *Proceedings*, Jan.-Feb. 1936 Journal.

Patents for License

The United States Patent office has put into operation a new service to industry and inventors, to bring to the attention of the public, patented inventions under which the owners are willing to grant licenses on reasonable terms. To accomplish the results desired by the new service, a "Register of Patents" is now being established and will be maintained in the United States Patent Office. It will be available to the public for inspection in Washington. Lists of patents will be published in the *Official Gazette* of the patent office.

Honor Roll

February 1 to May 31, 1945

Rene Morales, in Havana, Cuba heads the list with 11 new Members since Feb. 1.

Rene Morales	**11**
A. Amirikian	**3**
J. H. Spilkin	**3**
Charles S. Whitney	**3**
Ernst Gruenwald	**2½**
Charles E. Wuerpel	**2½**
C. Blaschitz	**2**
Francis MacLeay	**2**
J. M. Wells	**2**
J. W. Kelly	**1½**
H. W. Cormack	1
Harry B. Dickens	1
R. F. Dierking	1
Harrison F. Gonnerman	1
G. H. Hodgson	1
William G. McFarland	1
Denis Matthews	1
D. E. Parsons	1
A. F. Penny	1
Byram Steel	1
G. W. Stokes	1
M. A. Timlin	1
Maxwell Upson	1
Stanton Walker	1
Roy Zipprodt	1

The following credits are, in each instance, "50-50" with another Member:

Birger Arneberg	Adolph Meyer
E. W. Bauman	M. D. Olver
Harmer E. Davis	Kanwar Sain
L. I. Johnstone	J. L. Savage
H. J. McGillivray	Oskar Schreier
R. E. McLaughlin	Wm. T. Summers

W. F. Way Only Colonel in Navy

Col. W. F. Way (ACI Member since 1921) wrote us not long ago from England. He ventures that he has at least one distinction in being "the only Colonel in U. S. Navy." His address is Colonel W. F. Way, U. S. Navy, 949, c/o Fleet Post Office, New York City. In 27 months he says he's had eight addresses. His letter was written before V-E day and doesn't say too much, but he does report that he has been getting the ACI JOURNAL

lately and has read it in spite of the fact that destruction and not construction is his business. He says "I laugh at 'concrete for permanence' as an old fashioned idea, and if we are to continue to have wars, it's better to build with bricks because they can be used over again, while destroyed concrete structures don't seem to make even good fill".

Further he says, "really I'm in the Navy and assigned to ship salvage. The work has been very interesting; even if not too constructive at times." He reports having been through the Mediterranian and Normandy campaigns and recently in England to prepare for another duty.

Norman F. Stineman

one time editor of *Concrete*, Chicago, member of the Institute since 1930; twice a contributor to the Institute *Proceedings*, who has been connected with various federal war agencies in the engineering field for some time, is now in the Veterans Administration, identified with a big program of hospital construction.

Wallace L. Caldwell

ACI member since 1938, died April 22 in a Birmingham (Ala.) hospital from injuries received March 21 when his automobile rolled against him after he had parked it. He had been a resident of Birmingham since 1912 but was absent for some time in the twenties because of various executive responsibilities.

He first went to Birmingham as manager of the Pittsburgh Testing Laboratory, and at the time of his death, age 56, was president of the Alabama Asphaltic Limestone Co., president Southern Vacuum Concrete, Inc., president of the Aerocrete Western Corp., Chicago, and Vice Chairman of the Southern Research Institute.

He had been active in the field of lightweight concrete and lightweight concrete aggregates; was the inventor of the Caldwell slag machine. He had been active in civic and church affairs in Birmingham; was a past president of the Associated Industries of Alabama and the Rotary Club of Birmingham. At the time of his death he was a Director of the National Association of Manufacturers and of the Associated Industries of Alabama. He was also a member of the American Society of Civil Engineers, American Chemical Society, American Society for Testing Materials, American Society of Municipal Engineers, Association of Asphalt Paving Technologists, Alabama Engineers and Birmingham Engineer Club. He was a deacon of the Independent Presbyterian Church.

Journal Volume 16—
***Proceedings* Vol. 41**

This June issue of the ACI Journal concludes Journal V. 16, Proceedings V. 41 EXCEPT for the Supplement to the November 1945 Journal which will contain Title page, "Contents", closing discussion and Indexes—this Supplement to be mailed with the November Journal. Librarians watch for it. The Volume is then complete, ready to go to the binders.

SYNOPSES of RECENT ACI PAPERS and REPORTS are available for the asking - - - - below is a small sampling only from JOURNAL issues of the current volume year

A pamphlet including synopses for all of the current volume, Sept. 1944 to June 1945, will soon be available. The following and many others may be had in separate prints at 25 cents each except as otherwise noted; starred titles are 50 cents each.

THE EFFECTIVENESS OF VARIOUS TREATMENTS AND COATINGS FOR CONCRETE IN REDUCING THE PENETRATION OF KEROSENE..... 41-2

F. B. HORNIBROOK—Sept. 1944, p. 13-20 (V. 41)

Measurements were made of the penetration of kerosene under a 12 foot pressure head into discs of concrete which had received various treatments or coatings. The tests were classified into 7 groups as follows: (1) reference concrete, (2) integral admixtures, (3) sodium silicate and magnesium fluosilicate treatments, (4) plaster coats, plain and with admixtures, (5) magnesium oxychloride type coatings, (6) linseed oil and spar varnish coatings, and (7) synthetic plastic and latex coatings. Comparisons of the relative rates of penetration of kerosene into the specimens of each group are given, together with a discussion of various other properties of each group.

ADMIXTURES FOR CONCRETE..... 41-5

A REPORT BY ACI COMMITTEE 212—Nov. 1944, pp.73-88 (V. 41)

With the aim of providing a perspective of the field of admixtures for the use of the engineer confronted with a need of modifying concrete to meet special requirements of a given job, Committee 212 has classified admixtures into 9 broad groups. Discussions are given of the factors which might indicate the usefulness of admixtures of each group, and of the important effects which may ordinarily be expected from the use of materials of each group. The 9 groups are as follows: (1) accelerators, (2) air-entraining agents, (3) gas-forming agents, (4) natural cementing materials, (5) pozzolanic materials, (6) retarders, (7) water-repelling agents, (8) workability agents, and (9) miscellaneous.

THE EFFECT OF CURING CONDITIONS ON COMPRESSIVE, TENSILE AND FLEXURAL STRENGTH OF CONCRETE CONTAINING HAYDITE AGGREGATE..... 41-7

E. B. HANSON, JR. and W. T. NEELANDS—Nov. 1944, pp. 105-116 (V. 41)

Lightweight concrete has been given a severe test in the U. S. Maritime Commission's present concrete ship construction program. In its use, problems arose that could not be solved by the application of sand-gravel concrete data. This paper describes some of the strength characteristics of this type of concrete. Data herein are consistent in showing that rapid moisture loss from Haydite concrete produces a serious retrogression in the tensile and flexural strengths, regardless of the length of moist curing. This decline in strength, caused by drying shrinkage stresses developing in the outer fibers as the moisture content becomes unbalanced, is of a temporary nature and apparently can be curbed by the application of paint or membrane seal following the moist curing period. The drying shrinkage may well contribute to serious cracking in some types of structures if control is not maintained.

CONCRETE OPERATIONS IN THE CONCRETE SHIP PROGRAM★ 41-9

LEWIS H. TUTHILL—Jan. 1945, pp. 137-180 (V. 41)

This paper describes only briefly the hulls constructed in the concrete ship program of the U. S. Maritime Commission but goes into more detail in connection with problems encountered and their solution in the course of these concrete operations. Construction joint procedure, lightweight aggregate concrete and mix control, handling, placing and vibration practice, curing, testing and repair problems are described in the belief that much of this information is applicable to any concrete work of high standard. Design of hulls is not discussed except as construction is affected. The ships have not been in service sufficiently long to justify much discussion of their performance or durability.

AN INSTRUMENT AND A TECHNIC FOR FIELD DETERMINATION OF THE MODULUS OF ELASTICITY, AND FLEXURAL STRENGTH, OF CONCRETE (PAVEMENTS).....41-11

BARTLETT G. LONG, HENRY J. KURTZ, and THOMAS A. SANDENAW—Jan. 1945, pp. 217-232 (V. 41)

An instrument for determination of the dynamic modulus of elasticity of concrete, in situ, is described. Test results are presented which show (a) the comparison of test values of E, obtained by various older methods, with that obtained with the new instrument; and (b) the relationship of such values to the flexural strength of concrete. It is concluded that adoption of the new method and technic is justified, and that widespread use of the new instrument would eliminate the necessity for casting field specimens during construction (except perhaps for day-to-day control purposes) or of removing costly "samples" from completed works. A method for determining the thickness of concrete pavements is briefly discussed. A rather extensive bibliography is included.

synopses pamphlets, providing a survey of recent ACI work, are cheerfully sent on request - - - address

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Sources of Equipment, Materials and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 45th St., Alpena, Mich. 428
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 387
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment

Baily Vibrator Co., 1526 Wood St., Philadelphia 2, Pa. 407
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 394-5
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 421
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 418-9
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 410-11
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 389
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 377
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 378-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 392-3
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 429
—Mixing and batching plants, buckets, elevators

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 422-3
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 380
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 427
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 430-1
—Concrete vibrators

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Ransome Machinery Co., Dunellen, N. J. 433
—Mixers—paving, truck

Viber Co., 726 So. Flower St., Burbank, Calif. 381
—Concrete vibrators

Contractors, Engineers and Special Services

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 390-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 397-400
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 396
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 426
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 416
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 409
—Suction control of water in the concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 385
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge, Mass. 412-5
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 417
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 425
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 384
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 382-3
—Portland cements

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 402-5
—Cement dispersing and air-entraining agents

Rail Steel Bar Association 386
—Reinforcement bars

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 408
—Form tying devices

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 434-5
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 401
—Form lining

Testing Equipment

American Machine & Metals, Inc., East Moline, Ill. 388
—Riehle hydraulic testing machines

A few of the ACI special publications in large current demand

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members. $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information about ACI Membership and Publications address

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 41—1945

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 16, Sept. 1944, to June 1945 and Supplement, November 1945

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference to discussion is by supplementary page numbers. If the last page of a paper is 28, the first page of the discussion, published later, is 28 - 1.

Important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN and other subjects classified under this head, are indented. These samples do not apply fully to any single year's index.

In general, key words to important subjects appear in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The complete title of each paper is indexed. This title is repeated in full following the name of the author in its proper alphabetical order. Numerous cross references to parts of the subject matter covered, do not always indicate the full scope. This may be found by looking up the item referred to under the name of the author or the title of the paper.

The readiest use may be made of this index by gaining some familiarity with the main classifications as follows:

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BUILDING CODES
BUILDINGS
BUILDING UNITS
CENTRAL MIXING
COMMITTEES
CONSTRUCTION—DETAILS, METHODS AND COSTS
CURING
DAMS
DURABILITY
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FIRES, FIRE TESTS AND FIREPROOFING
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